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STRUCTURES TO RESIST THE EFFECTS OF ACCIDENTAL EXPLOSIONS
VOLUME VI, SPECIAL CONSIDERATIONS IN EXPLOSIVE FACILITY DESIGN

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This report is Volume VI of six volumes which will eventually be published as a tri-service design manual. The project was sponsored by the Department of Defense Explosives Safety Board.

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Structural design	Pre-engineered buildings	Shock isolation systems
Masonry walls	Suppressive shields	Underground structures
Precast concrete	Earth covered magazines	
Prestressed concrete	Arch-type magazines	
Blast resistant structures	Blast valves	

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

This report, in six volumes, details design procedures for structures which are subjected to the effects of accidental explosions. The procedures cover the determination of the blast environment (blast and fragments) and then structural design. This volume, "Special Considerations in Explosive Facility Design," in particular, contains procedures for the design of blast-resistant structures other than above ground, cast-in-place concrete or structural steel structures, as well as the design of other miscellaneous blast-resistant components.

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20. ABSTRACT (cont)

Included in this volume are the design of reinforced and nonreinforced masonry walls, precast elements both prestressed and conventionally reinforced, pre-engineered buildings, suppressive shielding, blast resistant windows, underground structures, earth covered, arch-type magazines, blast valves, and shock isolation systems.

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VOLUME VI
SPECIAL CONSIDERATIONS IN EXPLOSIVE FACILITY DESIGN

INTRODUCTION

6-1 Purpose

The purpose of this six volume manual is to present methods of design for protective construction used in facilities for development, testing, production, maintenance, modification, inspection, disposal and storage of explosive materials.

6-2 Objectives

The primary objectives are to establish design procedures and construction techniques whereby propagation of explosion (from one building or part of a building to another) or mass detonation can be prevented and protection for personnel and valuable equipment will be provided.

The secondary objectives are:

(1) Establish the blast load parameters required for design of protective structures;
(2) Provide methods for calculating the dynamic response of structural elements including reinforced concrete, structural steel, etc.;
(3) Establish construction details and procedures necessary to afford the required strength to resist the applied blast loads;
(4) Establish guidelines for siting explosive facilities to obtain maximum cost effectiveness in both the planning and structural arrangements; providing closures, and preventing damage to interior portions of structures due to structural motion, shock, and fragment perforation.

6-3 Background

For the first 60 years of the 20 Century criteria and methods based upon the results of catastrophic events have been used for the design of explosive facilities. The criteria and methods did not include a detailed or reliable quantitative basis for assessing the degree of protection afforded by the protective facility. In the late 1960's quantitative procedures were set forth in the first edition of the present manual, "Structures to Resist the Effects of Accidental Explosions." This manual was based on extensive research and development programs which permitted a more reliable approach to design requirements. Since the original publication of this manual, more extensive publication, more extensive testing and development programs have taken place. This additional research was directed primarily towards

materials other than reinforced concrete which was the principal construction material referenced in the initial version of the manual.

Modern methods for the manufacture and storage of explosive materials, which include many exotic chemicals, fuels, propellants, etc., required less space for a given quantity of explosive material than was previously needed. Such concentration of explosives increase the possibility of the progagation of accidental explosions (one accidental explosion causing the detonation of other explosive materials). It is evident that a requirement for more accurate design techniques has become essential. This manual describes rational design methods to provide the required structural protection.

These design methods account for the close-in effects of a detonation including associated high pressures and nonuniformity of the blast loading on protective structures or barriers as well as intermediate and far-range effects which are encountered in the design of structures which are positioned away from the explosion. The dynamic response of structures, constructed of various materials, or combination of materials, can be calculated, and details have been developed to provide the properties necessary to supply the required strength and ductility specified by the design. Development of these procedures has been directed primarily towards analyses of protective structures subjected to the effects of high explosive detonation. However, this approach is general and is applicable to the design of other explosive environments as well as other explosive materials as numerated above.

The design techniques set forth in this manual are based upon the results of numerous full- and small-scale structural response and explosive effects tests of various materials conducted in conjunction with the development of this manual and/or related projects.

6-4 Scope of Manual

This manual is limited only by variety and range of the assumed design situation. An effort has been made to cover the more probable situtations. However, sufficient general information on protective design techniques has been included in order that application of the basic theory can be made to situations other than those which were fully considered.

This manual is generally applicable to the design of protective structures subjected to the effects associated with high explosive detonations. For these design situations, this manual will generally apply for explosive quantities less than 25,000 pounds for close-in effects. However, this manual is also applicable to other situations such as far or intermediate range effects. For these latter cases the design procedures as presented are applicable for explosive quantities up to 500,000 pounds.

Because the tests conducted so far in connection with this manual have been directed primarily towards the response of structural steel and reinforced concrete elements to blast overpressures, this manual concentrates on design procedures and techniques for these materials. However, this does not imply that concrete and steel are the only useful materials for protective construction. Tests to establish the response of wood, brick blocks, plastics, etc. as well as the blast attenuating and mass effects of soil are

contemplated. The results of these tests may require, at a later date, the supplementation of these design methods for these and other materials.

Other manuals are available which enable one to design protective structures against the effects of high explosive or nuclear detonations. The procedures in these manuals will quite often complement this manual and should be consulted for specific applications.

Computer programs, which are consistent with the procedures and techniques contained in the manual, have been apporoved by the appropriate representative of the U.S. Army, the U.S. Navy, the U.S. Air Force and the Deparment of Defense Explosive Safety Board (DDESB). These programs are available through the following repositories:

1. Department of the Army

   Commander and Director
   U.S. Army Engineer
   Waterways Experiment Station
   Post Office Box 631
   Vicksburg, Mississippi 39180

   Attn: WESKA

2. Department of the Navy

   Officer-In-Charge
   Civil Engineering Laboratory
   Naval Battalion Construction Center
   Port Hueneme, California 93043

   Attn: Code L51

3. Department of the Air Force

   Aerospace Structures
   Information and Analysis Center
   Wright Paterson Air Force Base
   Ohio 45433

   Attn: AFFDL/FBR

Limited number of copies of the above program are available from each repository upon request. The individual programs are identical at each repository. If any modifications and/or additions to these programs are required, they will be submitted by the organization for review by DDESB and the above services. Upon concurrence of the revisions, the necessary changes will be made and notification of these changes will be made by the individual repositories.

6-5 Format of Manual

This manual entitled, "Structures to Resist the Effects of Accidental Explosions," is subdivided into six specific volumes dealing with various aspects of design. The titles of these volumes are as follows:

Volume	I	-	Introduction
Volume	II	-	Blast, Fragment and Shock Loads
Volume	III	-	Principles of Dynamic Analysis
Volume	IV	-	Reinforced Concrete Design
Volume	V	-	Structural Steel Design
Volume	VI	-	Special Considerations in Explosive Facility Design

Appendix A pertinent to a particular volume and containing illustrative examples of the explosive effects and structural response problems appear at the end of each volume.

Commonly accepted symbols have been used as much as possible. However protective design involves many different scientific and engineering fields, and, therefore, no attempt has been made to standardize completely all the symbols used. Each symbol has been defined where it is first introduced, and a list of the symbols, with their definitions and units, is contained in Appendix B of each volume.

VOLUME CONTENTS

6-6 General

This volume contains procedures for the design of blast resistant structures other than above ground, cast-in-place concrete or structural steel structures, as well as the design of other miscellaneous blast resistant components. Included herein is the design of reinforced and non-reinforced masonry walls, precast elements both prestressed and conventionally reinforced, pre-engineered buildings, suppressive shielding, blast resistant windows, underground structures, earth-covered arch-type magazines, blast valves and shock isolation systems.

MASONRY

6-7 Application

Masonry units are used primarily for wall construction. These units may be used for both exterior walls subjected to blast overpresures and interior walls subjected to inertial effects due to building motions. Basic variations in wall configurations may be related to the type of masonry unit such as brick, clay tile or solid and hollow concrete masonry units (CMU), and the manner in which these units are laid (running bond, stack bond, etc.), the number of wythes of units (single or double), and the basic lateral load-carrying mechanism (reinforced or non-reinforced, one or two-way elements).

In addition to their inherent advantages with respect to fire protection, acoustical and thermal insulation, structural mass and resistance to flying debris, masonry walls when properly designed and detailed can provide economical resistance to relatively low blast pressures. However, the

limitation on their application includes a limited capability for large deformations, reduced capacity in rebound due to tensile cracking in the primary phase of the response as well as the limitations on the amount and type of reinforcement which can be provided. Because of these limitations, masonry construction in this Manual is limited to concrete masonry unit (CMU) walls placed in a running bond and with single or multiple wythes. However, because of the difficulty to achieve the required interaction between the individual wythes, the use of multiple wythes should be avoided.

Except for small structures (such as tool sheds, garage, etc.) where the floor area of the building is relatively small and interconnecting block walls can function as shear walls, masonry walls will usually require supplementatry framing to transmit the lateral forces produced by the blast forces to the building foundation. Supplementary framing is generally classified into two categories (depending on the type of construction used); namely (1) flexible type supports such as structural steel framing, and (2) rigid supports including reinforced concrete frames or shear wall slab construction. The use of masonry walls in combination with structural steel frames is usually limited to incident over-pressures of 2 psi or less while masonry walls when supported by rigid supports may be designed to resist incident pressures as high as 10 psi. Figures 6-1 and 6-2 illustrate these masonry support systems.

Depending on the type of construction used, masonry walls may be classified into three categories: namely (1) cavity walls, (2) solid walls, and (3) a combination of cavity and solid walls. The cavity walls utilize hollow load-bearing concrete masonry units conforming to ASTM C90. Solid walls use solid load-bearing concrete masonry units conforming to ASTM C145 or hollow units whose cells and voids are filled with grout. The combined cavity and solid walls utilize the combination of hollow and solid units. Masonry walls may be subdivided further depending on the type of load-carrying mechanism desired: (1) joint reinforced masonry construction, (2) combined joint and cell reinforced masonry construction, and (3) non-reinforced masonry construction.

Joint reinforced masonry construction consists of single or multiple wythes walls and utilizes either hollow or solid masonry units. The joint reinforced wall construction utilizes commercially available cold drawn wire assemblies (see fig. 6-3), which are placed in the bed joints between the rows of the masonry units. Two types of reinforcement are available; truss and ladder types. The truss reinforcement provides the more rigid system and, therefore, is recommended for use in blast resistance structures. In the event that double wythes are used, each wythe must be reinforced independently. The wythes must also be tied together using wire ties. Joint reinforced masonry construction is generally used in combination with flexible type supports. The cells of the units located at the wall supports must be filled with grout. Typical joint reinforced masonry construction are illustrated in figure 6-4.

Combined joint and cell reinforcement masonry construction consists of single wythe walls which utilize both horizontal and vertical reinforcement. The horizontal reinforcement may consist either of the joint reinforcement previously discussed or reinforcing bars. Where reinforcing bars are used, special masonry units are used which permit the reinforcement to sit below the joint (fig. 6-5). The vertical reinforcement consists of reinforcing bars which are positioned in one or more of the masonry units cells. All cells,

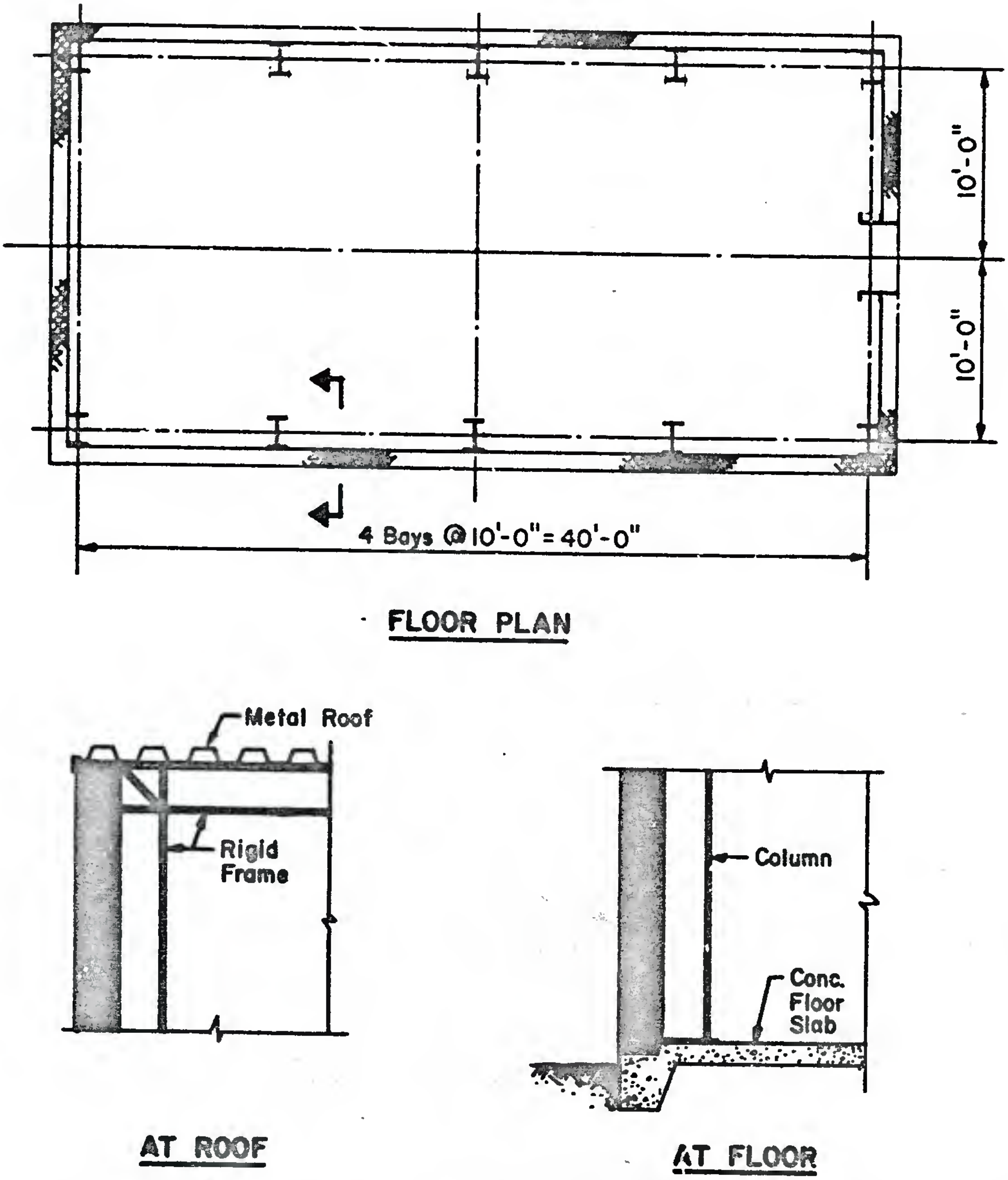
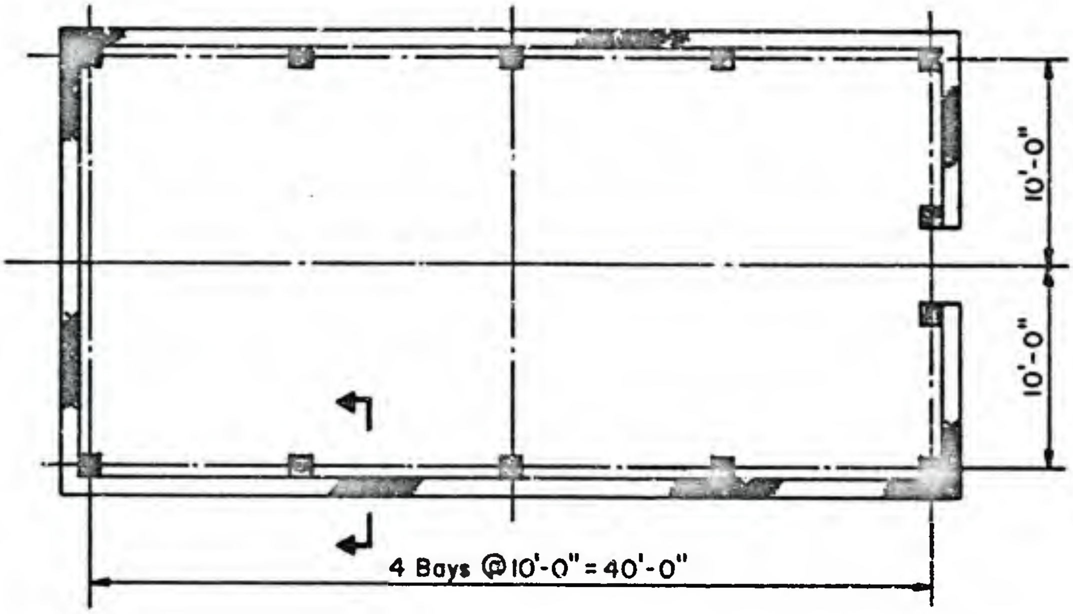
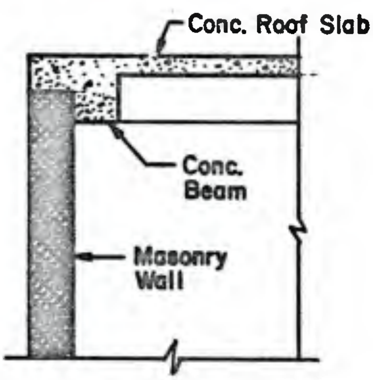


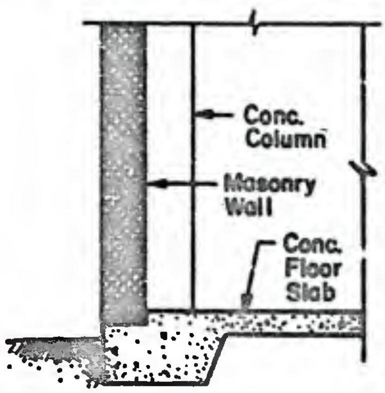
Figure 6-1 Masonry wall with flexible support



FLOOR PLAN



AT ROOF



AT FLOOR

Figure 6-2 Masonry wall with rigid support

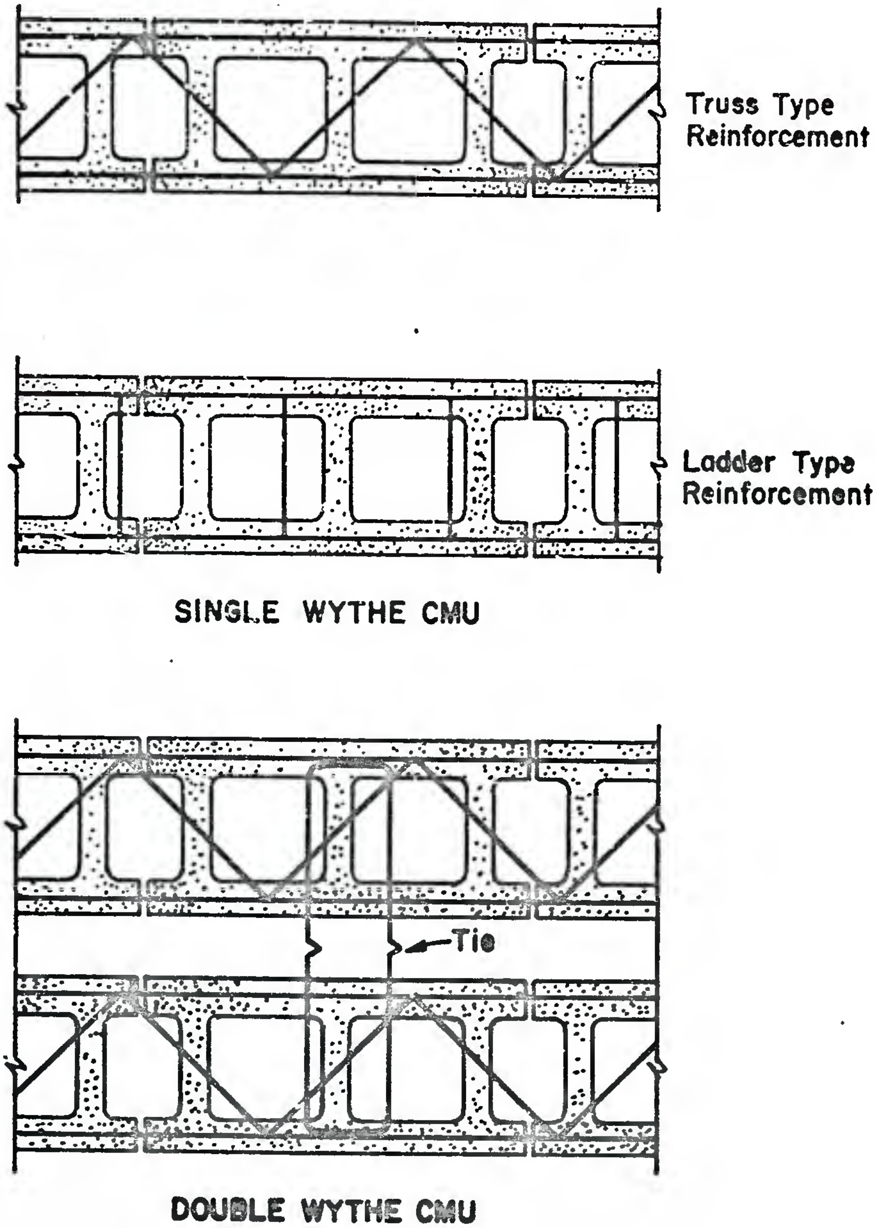
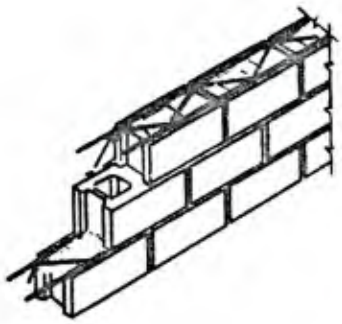
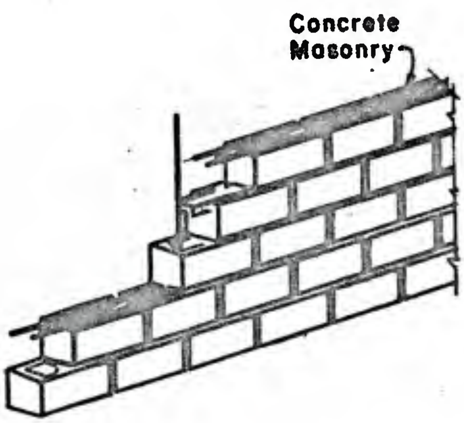


Figure 6-3 Concrete masonry walls



JOINT REINFORCED MASONRY CONSTRUCTION



COMBINED JOINT AND CELL REINFORCED
MASONRY CONSTRUCTION

Figure 6-4 Typical joint reinforced masonry construction

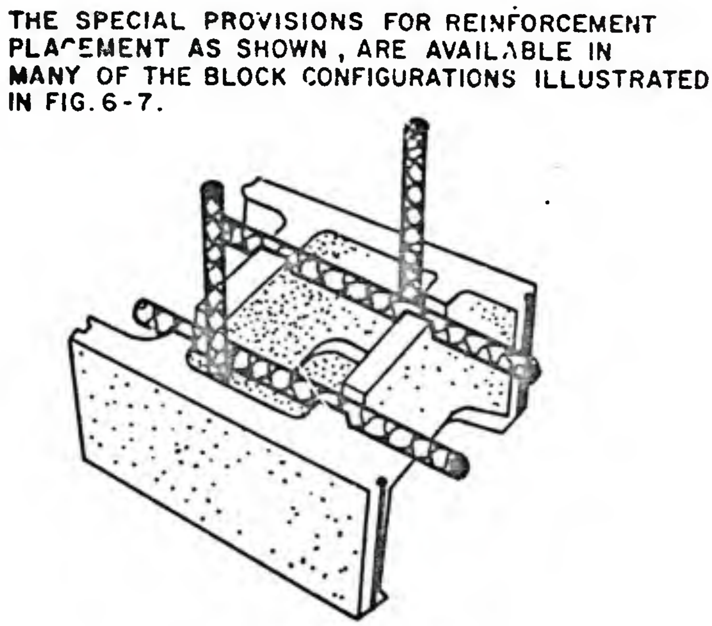


Figure 6-5 Special masonry unit for use with reinforcing bars

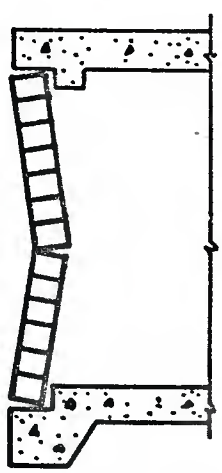


Figure 6-6 Arching action of non-reinforced masonry wall

which contain reinforcing bars, must be filled with grout. Depending on the amount of reinforcement used, this type of construction may be used with either the flexible or rigid type support systems.

Non-reinforced masonry construction consists of single wythe of hollow or solid masonry units. This type of construction does not utilize reinforcement for strength but solely relies on the arching action of the masonry units formed by the wall deflection and support resistance (fig. 6-6). This form of construction is utilized with the rigid type support system and, in particular, the shear wall and slab construction system.

6-8 Design Criteria for Reinforced Masonry Walls

6-8.1 *Static Capacity of Reinforced Masonry Units*

Figure 6-7 illustrates typical shapes and sizes of concrete masonry units which are commercially available. Hollow masonry units shall conform to ASTM C90, Grade N. This grade is recommended for use in exterior below and above grade and for interior walls. The minimum dimensions of the components of hollow masonry units are given in table 6-1.

Table 6-1 Properties of Hollow Masonry Units

Nominal Width of Units (in)	Face-Shell Thickness (in)	Equiv. Web Thickness (in)
3 and 4	0.75	1.625
6	1.00	2.25
8	1.25	2.25
10	1.375	2.50
12	1.500	2.50

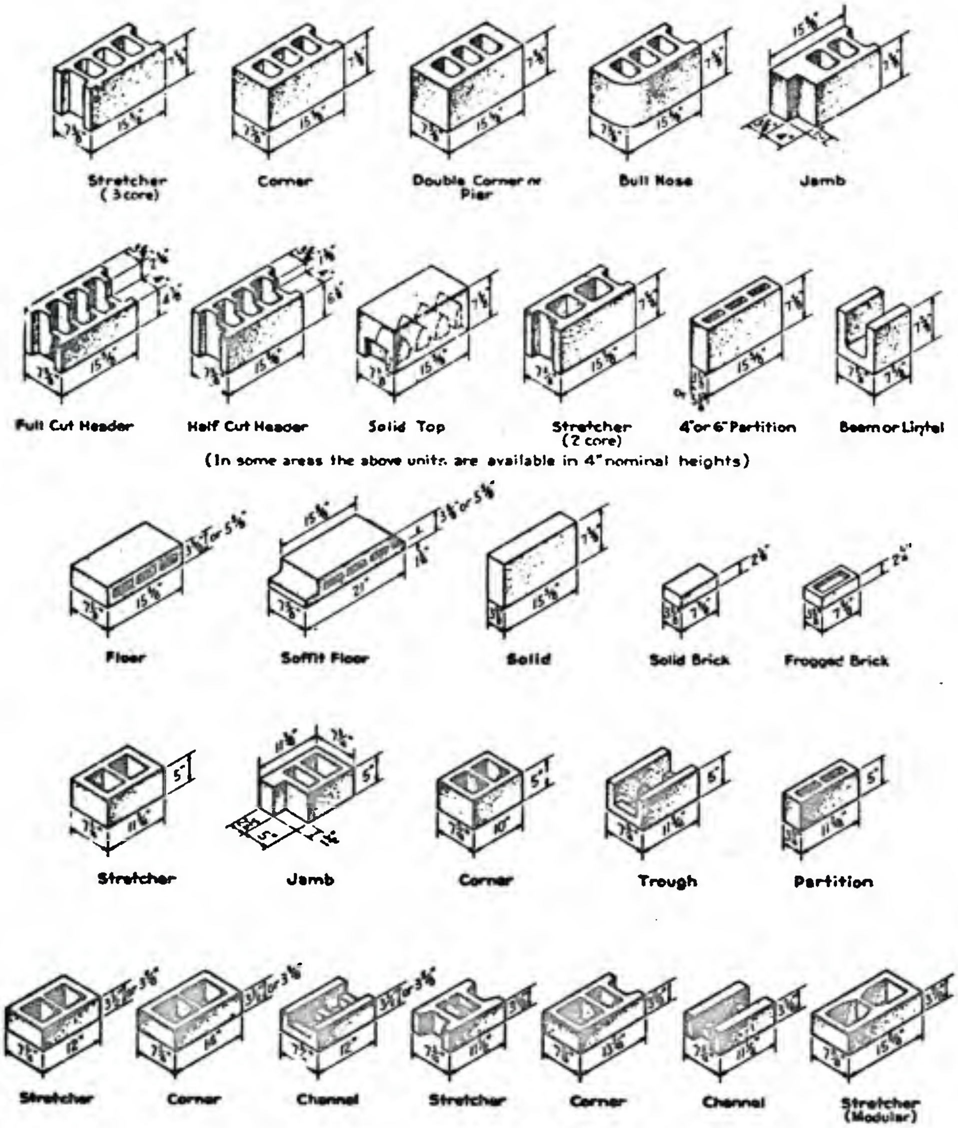


Figure 6-7 Typical concrete masonry units

The specific compressive strength (f'_m) for concrete masonry units may be taken as:

Type of Unit	Ultimate Strength (f'_m)
Hollow Units	1350 psi
Hollow Units filled with grout	1500 psi
Solid Units	1800 psi

while the modulus of elasticity (E_m) of masonry units is equal to:

$$E_m = 1000\ f'_m \qquad 6\text{-}1$$

The specific compressive strength and the modulus of elasticity of the mortar may be assumed to be equal to that of the unit.

Joint reinforcement shall conform to the requirements of ASTM A82 and, therefore, it will have a minimum ultimate (f_u) and yield (f_y) stresses equal to 80 ksi and 70 ksi respectively. Reinforcing bars shall conform to ASTM A615 (Grade 60) and have minimum ultimate stress (f_u) of 90 ksi and minimum yield stress (f_y) of 60 ksi. The modulus of elasticity of the reinforcement is equal to 29,000,000 psi.

6-8.2 *Dynamic Strength of Material*

Since design for blast resistant structures is based on ultimate strength, the actual yield stresses of the material, rather than conventional design stresses or specific minimum yield stresses, are used for determining the plastic strengths of members. Further, under the rapid rates of straining that occur in structures loaded by blast forces, materials develop higher strengths than they do in the case of statically loaded members. In calculating the dynamic properties of concrete masonry construction it is recommended that the dynamic increase factor be applied to the static yield strengths of the various components as follows:

Concrete	
Flexure	$1.19\ f'_m$
Shear	$1.00\ f'_m$
Compression	$1.12\ f'_m$
Reinforcement	
Flexure	$1.17\ f_y$

6-8.3 *Ultimate Strength of Reinforced Concrete Masonry Walls*

The ultimate moment capacity of joint reinforced masonry construction may be conservatively estimated by utilizing the horizontal reinforcement only and neglecting the compressive strength afforded by the concrete. That is the reinforcement in one face will develop the tension forces while the steel in the opposite face resists the compression stresses. The ultimate moment relationship may be expressed for each horizontal joint of the wall as follows:

$$M_u = A_s f_{dy} d_c \qquad 6\text{-}2$$

where:

- A_s = area of joint reinforcement at one face
- f_{dy} = dynamic yield strength of the joint reinforcement
- d_c = distance between the centroids of the compression and tension reinforcement
- M_u = ultimate moment capacity

On the contrary, the ultimate moment capacity of the cell reinforcement (vertical reinforcement) in a combined joint and cell reinforced masonry construction utilizes the concrete strength to resist the compression forces.

The method of calculating ultimate moment of the vertical reinforcement is the same as that presented in Volume IV of this manual which is similar to that presented in the American Concrete Institute Standard Building Code Requirements for Reinforced Concrete.

The ultimate shear stress in joint reinforced masonry walls is computed by the formula:

$$v_u = V_u / A_n \qquad 6\text{-}3$$

where:

- v_u = unit shear stress
- V_u = total applied design shear at $d_c/2$ from the support
- A_n = net area of section

In all cases, joint reinforced masonry walls, which are designed to resist blast pressures, shall utilize shear reinforcement which shall be designed to carry the total shear stress. Shear reinforcement shall consist of; (1) bars or stirrups perpendicular to the longitudinal reinforcement, (2) longitudinal bars bent so that the axis or inclined portion of the bent bar makes an angle of 45 degrees or more with the axis of the longitudinal part of the bar; or (3) a combination of (1) and (2) above. The area of the shear reinforcement placed perpendicular to the flexural steel shall be computed by the formula:

$$A_v = \frac{v_u b s}{\phi f_y} \qquad 6\text{-}4$$

where:

A_v = area of shear reinforcement
b = unit width of wall
s = spacing between stirrups
f_y = yield stress of the shear reinforcement
ϕ = strength reduction factor equal to 0.85

When bent or inclined bars are used, the area of shear reinforcement shall be calculated using:

$$A_v = \frac{v_u bs}{\phi f_y\ (\sin \alpha + \cos \alpha)} \qquad 6\text{-}5$$

where:

α = angle between inclined stirrup and longitudinal axis of the member.

Shear reinforcement in walls shall be spaced so that every 45 degree line extending from mid depth ($d_c/2$) of a wall to the tension bars, crosses at least one line of shear reinforcement.

Cell reinforced masonry walls essentially consist of solid concrete elements. Therefore, the relationships, for reinforced concrete as presented in Volume IV of this manual may also be used to determine the ultimate shear stresses in cell reinforced masonry walls. Shear reinforcement for cell reinforced walls may only be added to the horizontal joint similar to joint reinforced masonry walls.

6-8.4 *Dynamic Analysis*

The principles for dynamic analysis of the response of structural elements to blast loads are presented in Volume III of this manual. These principles also apply to blast analyses of masonry walls. In order to perform these analyses, certain dynamic properties must be established as follows:

Load-mass factors, for masonry walls spanning in either one direction (joint reinforced masonry construction) or two directions (combined joint and cell reinforced masonry construction) are the same as those load-mass factors which are listed in tables 3-12 and 3-13. The load-mass factors are applied to the actual mass of the wall. The weights of masonry wall can be determined based on the properties of hollow masonry units previously described and utilizing a concrete unit weight of 150 pounds per cubic foot. The values of the load-mass factors K_{LM}, will depend in part on the range of behavior of the wall; i.e., elastic, elasto-plastic, and plastic ranges. An average value of the elastic and elasto-plastic value of K_{LM} is used for the elasto-plastic range while an average value of the average K_{LM} for the elasto-plastic range and K_{LM} of the plastic range is used for the wall behavior in the plastic range.

The resistance-deflection function is illustrated in figure 3-1. This figure illustrates the various ranges of behavior previously discussed and defines the relationship between the wall's resistances and deflections as well as

presents the stiffness K in each range of behavior. It may be noted in figure 3-1, that the elastic and elasto-plastic ranges of behavior have been idealized forming a bilinear (or trilinear) function. The equations for defining these functions are presented in Section 3-13.

The ultimate resistance r_u, of a wall varies; (1) as the distribution of the applied load, (2) geometry of the wall (length and width), (3) the amount and distribution of the reinforcement, and (4) the number and type of supports. The ultimate resistances of both one and two-way spanning walls are given in Section 3-9.

Recommended maximum deflection criteria for masonry walls subjected to blast loads is presented in table 6-2. This table includes criteria for both reusable and non-reusable conditions as well as criteria for both one and two-way spanning walls.

Table 6-2 Deflection Criteria for Masonry Walls

Wall Type	Support Type	Support Rotation
Reusable	One-way	0.5°
	Two-way	0.5°
Non-Reusable	One-way	1.0°
	Two-way	2.0°

When designing masonry walls for blast loads using response chart procedures of Volume III the effective natural period of vibration is required. This effective period of vibration when related to the duration of the blast loading of given intensity and a given resistance of the masonry wall determines the maximum transient deflection X_m of the wall. The expression for the natural period of vibration is presented in equation 3-60, where the effective unit mass m_e has been described previously and the equivalent unit stiffness K_E is obtained from the resistance - deflection function. The equivalent stiffness of one way beams is presented in table 3-8. This table may be used for one way spanning walls except that a unit width shall be used. Methods for determining the stiffneses and period of vibrations for two-way walls are presented in Sections 3-11 through 3-13.

Determining the stiffness in the elastic and elasto-plastic range is complicated by the fact that the moment of inertia of the cross section along the masonry wall changes continually as cracking progresses, and further by the fact that the modulus of elasticity changes as the stress increases. It is recommended that computations for deflections and therefore, stiffnesses be based on average moments of inertia I_a as follows:

$$I_a = \frac{I_n + I_c}{2} \qquad 6\text{-}6$$

In Equation 6-6, I_n is the moment of inertia of the net section and I_c is the moment of inertia of the cracked section. For solid masonry units the value of I_n is replaced with the moment of inertia of the gross section. The values of I_n and I_g for hollow and solid masonry units used in joint reinforced masonry construction are listed in table 6-3. The values of I_g for solid units may also be used for walls which utilize combined joint and cell masonry construction. The values of I_c for both hollow and solid masonry construction may be obtained using:

$$I_c = 0.005\ bd_c^3 \qquad 6\text{-}7$$

Table 6-3 Moment of Inertia of Masonry Walls

Type of Unit	Width of Unit (in)	Moment of Inertia (in^4)
Hollow	3	2.0
	4	4.0
	6	12.7
	8	28.8
	10	51.6
	12	83.3
Solid	3	2.7
	4	5.3
	6	18.0
	8	42.7
	10	83.0
	12	144.0

6-8.5 *Rebound*

Vibratory action of a masonry wall will result in negative deflections after the maximum positive deflection has been attained. This negative deflection is associated with negative forces which will require tension reinforcement to be positioned at the opposite side of the wall from the primary reinforcement. In addition, wall ties are required to assure that the wall is supported by the frame (fig. 6-8). The rebound forces are a function of the maximum resistance of the wall as well as the vibratory properties of the wall and the load duration. The maximum elastic rebound of a masonry wall may be obtained from figure 3-268.

6-9 Non-Reinforced Masonry Walls

The resistance of non-reinforced masonry walls to lateral blast loads is a function of the wall deflection, mortar compression strength and the rigidity of the supports.

6-9.1 *Rigid Supports*

If the supports are completely rigid and the mortar's strength is known, a resistance function can be constructed in the following manner.

Both supports are assumed to be completely rigid and lateral motion of the top and bottom of the wall is prevented. An incompletely filled joint is assumed to exist at the top as shown in figure 6-9a. Under the action of the blast load the wall is assumed to crack at the center. Each half then rotates as a rigid body until the wall takes the position shown in figure 6-9b. During the rotation the midpoint m has undergone a lateral motion X_c in which no resistance to motion will be developed in the wall, and the upper corner of the wall (point o) will be just touching the upper support. The magnitude of X_c can be found from the geometry of the wall in its deflected position:

$$(T-X_c)^2 = (L)^2 - [h/2 + (h'-h)/2]^2 \qquad 6\text{-}8$$

$$= (L)^2 - (h'/2)^2$$

where:

$$X_c = T - [(L)^2 - (h'/2)^2]^{1/2} \qquad 6\text{-}9$$

and

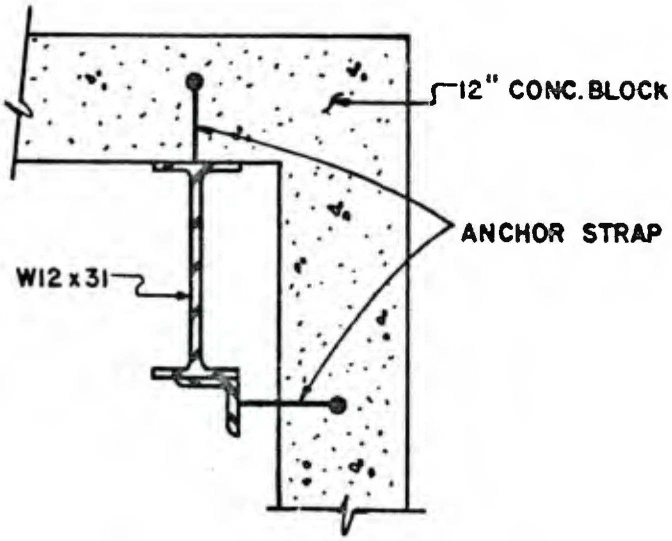
$$L = [(h/2)^2 + T^2] \qquad 6\text{-}10$$

All other symbols are shown in figure 6-9.

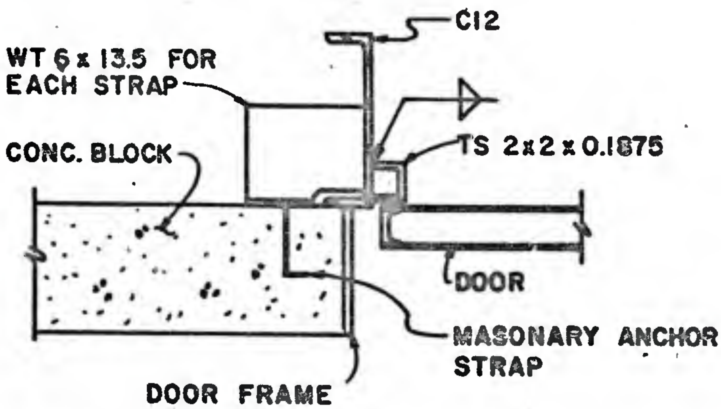
For any further lateral motion of point m, compressive forces will occur at points m and o. These compressive forces form a couple that produces a resistance to the lateral load equal to:

$$r_u = 8M_u/h^2 \qquad 6\text{-}11$$

where all symbols have previously been defined.



(a) Masonry Anchor Straps at Corners



(b) Masonry Anchor Strap Detail at Door.

Figure 6-8 Connection details for rebound and/or negative overpressures

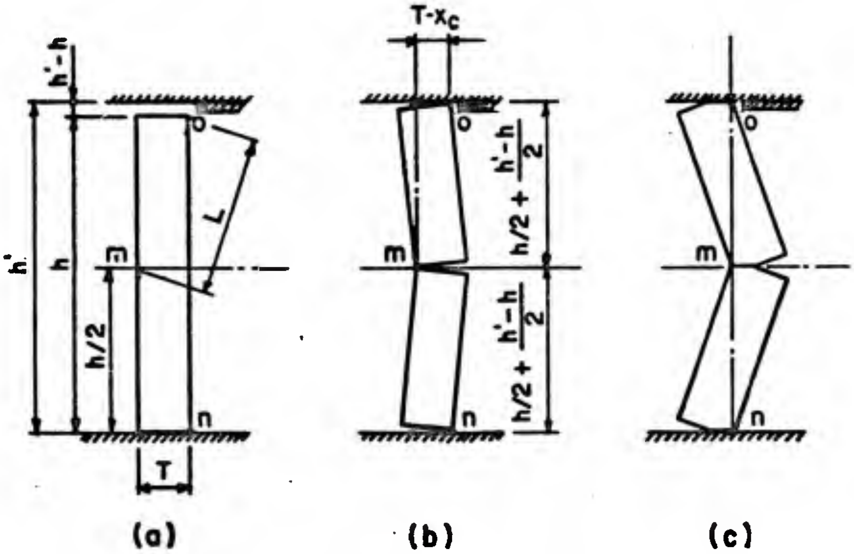


Figure 6-9 Deflection of non-reinforced masonry walls

When point m deflects laterally to a line n-o (fig. 6-9c), the moment arm of the resisting couple will be reduced to zero and the wall will become unstable with no further resistance to deflection. In this position the diagonals o-m and m-n will be shortned by an amount:

$$L - h'/2 \tag{6-12}$$

The unit strain in the wall caused by the shortening will be:

$$\varepsilon_m = (L - h'/2)L \tag{6-13}$$

where:

ε_m = unit strain in the mortar

All the shortening is assumed to occur in the mortar joints and therefore:

$$f_m = E_m \varepsilon_m \tag{6-14}$$

where:

E_m = modulus of elasticity of the mortar
f_m = compressive stress corresponding to the strain ε_m

In most cases f_m will be greater than the ultimate compressive strength of the mortar f'_m, and therefore cannot exist. Since for walls of normal height and thickness each half of the wall undergoes a small rotation to obtain the position shown in figure 6-9c, the shortening of the diagonals o-m and m-n can be considered a linear function of the lateral displacement of point m. The deflection at maximum resistance X_1, at which a compressive stress f'_m exists at points m, n and o can therefore be found from the following:

$$\frac{X_1 - X_c}{T - X_c} = \frac{f'_m}{f_m} = \frac{f'_m}{E_m \varepsilon_m} \tag{6-15a}$$

or

$$X_1 = \frac{(T - X_c)f'_m}{(E_m \varepsilon_m)} + X_c \tag{6-15b}$$

The resisting moment that is caused by a lateral deflection X_1 is found by assuming rectangular compression stress blocks to exist at the supports (points o and n) and at the center (point m) as shown in figure 6-10a. The bearing width a is chosen so that the moment M_u is a maximum, that is, by differentiating M_u with respect to a and setting the derivative equal to zero, which for a solid masonry unit will result in:

$$a = 0.5\ (T - X_1) \tag{6-16}$$

and the corresponding ultimate moment and resistance (fig. 6-10b) are equal to:

$$M_u = 0.25\ f'_m (T - X_1)^2 \tag{6-17}$$

and

$$r_u = (2/h^2) f'_m (T - X_1)^2 \tag{6-18}$$

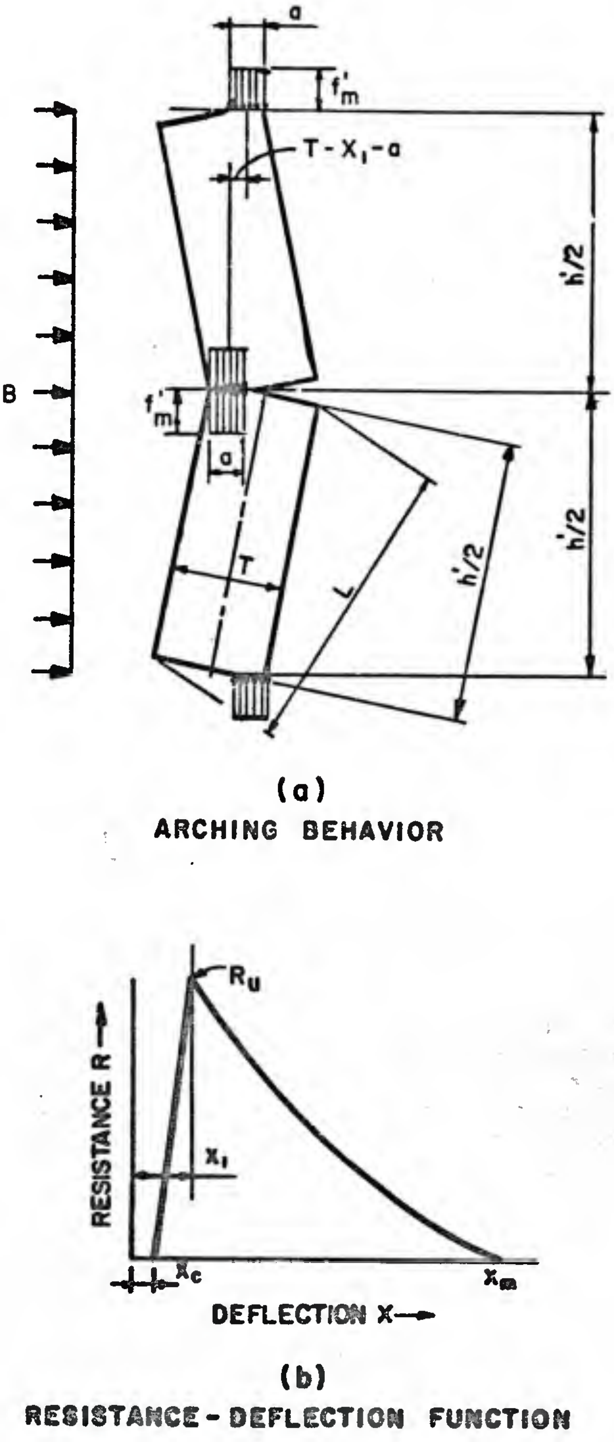


Figure 6-10 Structural behavior of non-reinforced solid masonry panel with rigid supports

When the mid span deflection is greater than X_1 the expression for the resistance as a function of the displacement is:

$$r = (2/h^2)f'_m(T - X)^2 \qquad 6\text{-}19$$

As the deflection increases the resistance is reduced until r is equal to zero and maximum deflection X_m is reached (fig. 6-10b). Similar expressions can be derived for hollow masonry units. However, the maximum value of a can not exceed the thickness of the flange width.

6-9.2 *Non-rigid Supports*

For the case where the wall is supported by elastic supports at the top and/or bottom, the resistance curve cannot be constructed based on the value of the compression force (af'_m) which is determined solely on geometry of the wall. Instead the resistance curve is a function of the stiffness of the supports. Once the magnitude of the compression force is determined, equations similar to those derived for the case of the rigid supports can be used.

6-9.3 *Simply Supported Walls*

If the supports offer no resistance to vertical motion, the compression in the wall will be limited by the wall weight above the floor plus any roof load which may be carried by the wall. If the wall carries no vertical loads, then the wall must be analyzed as a simply supported beam, the maximum resisting moment being determined by the modulus of rupture of the mortar.

PRECAST CONCRETE

6-10 Applications

Precast concrete construction can consist of either prestressed or conventionally reinforced members. Prestressing is advantageous in conventional construction, for members subjected to high flexural stresses such as long span or heavily loaded slabs and beams. Other advantages of precast concrete construction include: (1) completion time for precast construction will be significantly less than the required for cast-in-place concrete, (2) precast construction will provide protection against primary and secondary fragments not usually afforded by steel construction and (3) precast work is generally more economical than cast-in-plase concrete construction especially when standard precast shapes can be used. The overriding disadvantage of precast construction is that the use of precast members is limited to buildings located at relatively low pressure levels of 1 to 2 psi. For slightly higher pressure levels, cast-in-place concrete or structural steel construction becomes the more economical means of construction. However, for even higher pressures, cast-in-place concrete is the only means available to economically withstand the applied load.

Precast structures are of the shear wall type, rigid frame structures being economically impractical (see the discussion of connections, Section 6-16 below). Conventionally designed precast structures may be multi-story, but for blast design it is recommended that they be limited to single story buildings. Some of the most common precast sections are shown in figure 6-11. The single tee and double tee sections are used for wall panels and

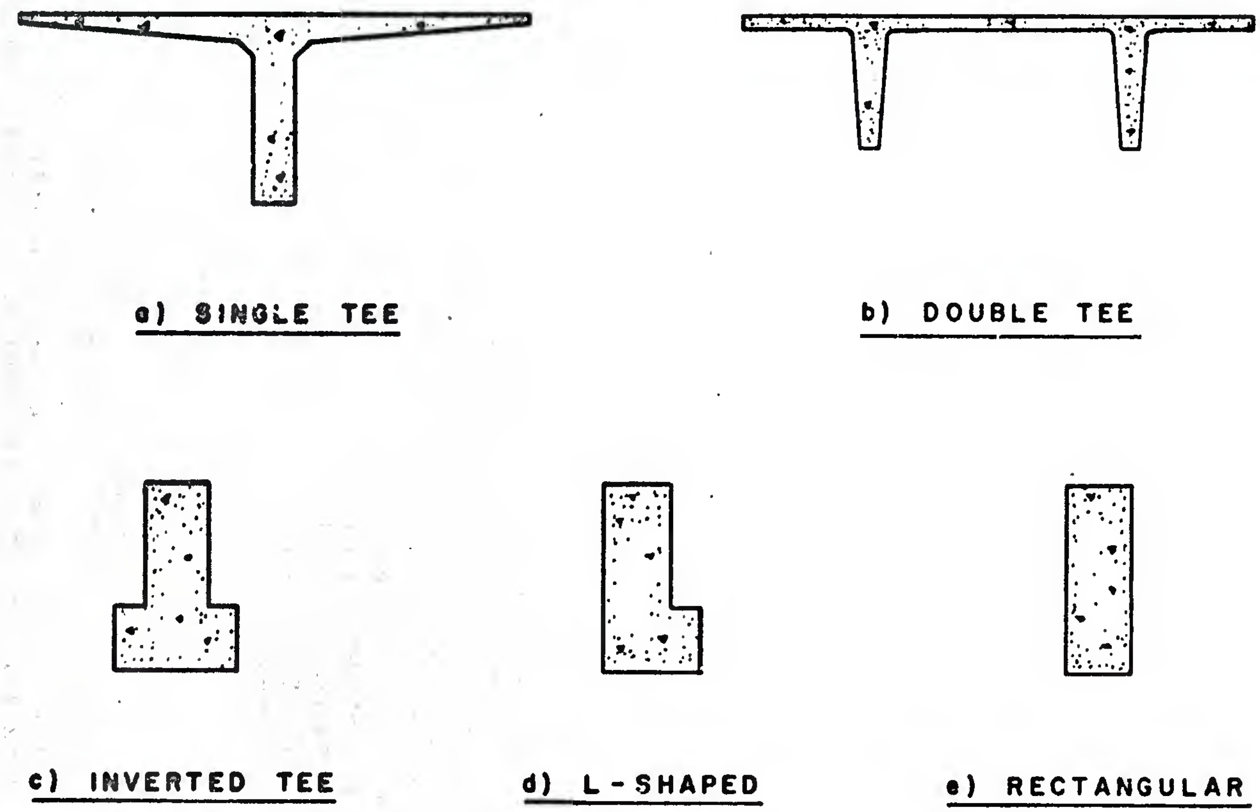


Figure 6-11 Common precast elements

roof panels. All the other sections are beam and girder elements. In addition, a modified flat slab section will be used as a wall panel around door openings. All of the sections shown can be prestressed or conventionally reinforced. In general though, for blast design, beams and roof panels are prestressed, while columns and wall panels are not. For conventional design, prestressing wall panels and columns is advantageous in tall multi-story building, and thus of no benefit for blast resistant design which uses only single story buildings. In fact, in the design of a wall panel, the blast load is from the opposite direction of conventional loads and hence prestressing a wall panel decreases rather than increases the capacity of section.

6-11 Static Strength of Materials

6-11.1 *Concrete*

Generally the minimum compressive strength of the concrete, f_c, used in precast elements is 4000 to 5000 psi. High early-strength cement is usually used in prestressed elements to ensure adequate concrete strength is developed before the prestress is introduced.

6-11.2 *Reinforcing Bars*

Steel reinforcing bars are used for rebound and shear reinforcement in prestressed members as well as for flexural reinforcement in non-prestressed members. For use in blast design, bars designated by the American Society for Testing and Materials (ASTM) as A 615, grade 60, are recommended. As only small deflections are permitted in precast members, the reinforcement is not stressed into its strain hardening region and thus the static design strength of the reinforcement is equal to its yield stress (f_y = 60,000 psi).

6-11.3 *Welded Wire Fabric*

Welded wire fabric, designated as A 185 by ASTM, is used to reinforce the flanges of tee and double tee sections. In conventional design welded wire fabric is sometimes used as shear reinforcement, but it is not used for blast design, which requires closed ties. The static design strength f_y, of welded wire fabric is equal to its yield stress, 65,000 psi.

6-11.4 *Prestressing Tendons*

There are several types of reinforcement that can be used in prestressing tendons. They are designated by ASTM as A 416, A 421 or A 722, with A 416, grade 250 or grade 270, being the most common. The high strength steel used in these types of reinforcement can only undergo a maximum elongation of 3.5 to 4 percent of the original length before the ultimate strength is reached. Furthermore, the high strength steel lacks a well defined yield point, but rather exhibits a slow continuous yielding with a curved stress-strain relationship until ultimate strength is developed (see fig. 6-12). ASTM specifies a fictitious yield stress f_{py}, corresponding to a 1 percent elongation. The minimum value of f_{py} depends on the ASTM designation, but it ranges from 80 to 90 percent of the ultimate strength, f_{pu}.

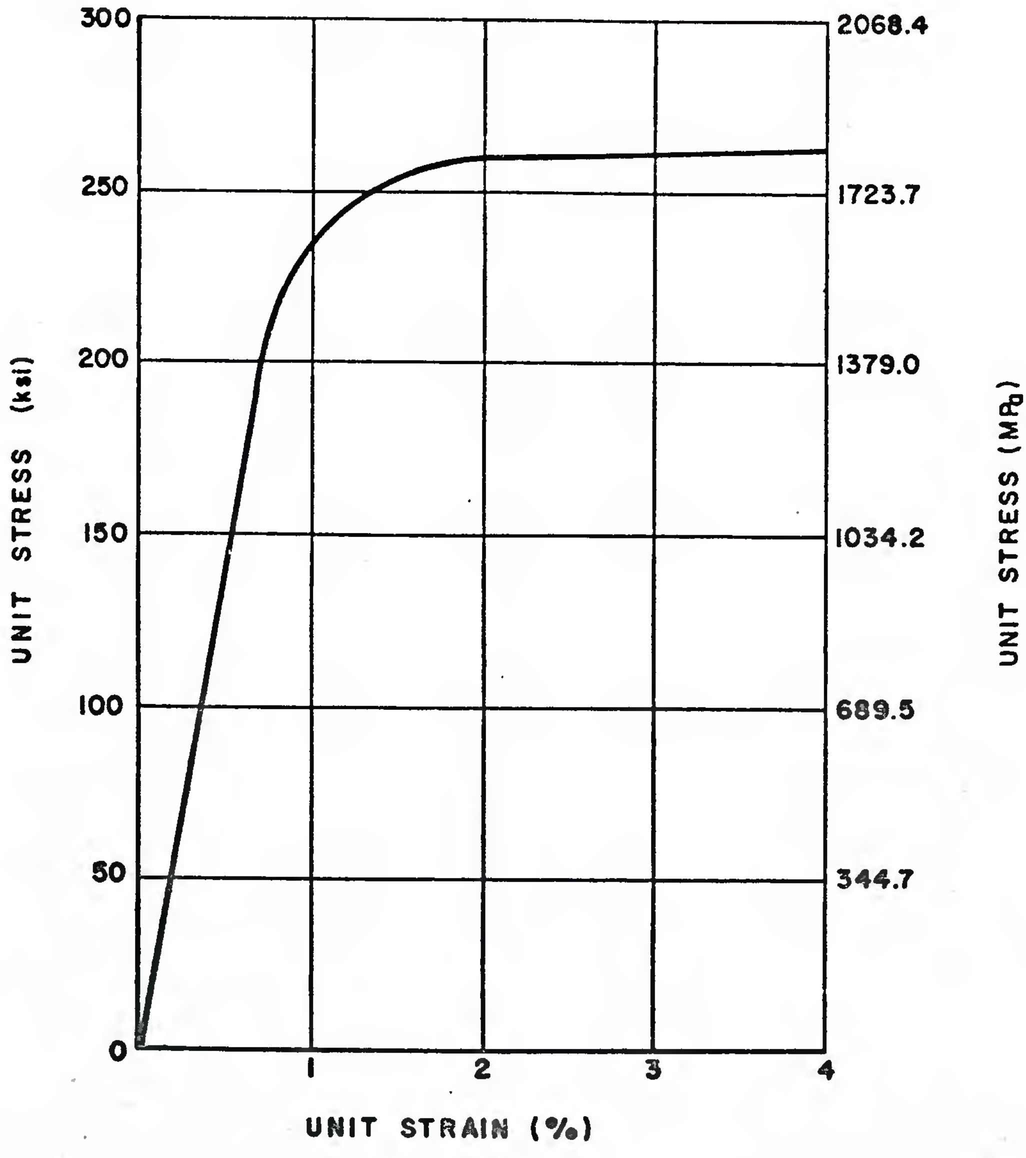


Figure 6-12 Typical stress-strain curve for high strength wire

6-12 Dynamic Strength of Materials

Under the rapid rate of straining of blast loads, most materials develop higher strengths than they do when statically loaded. An exception, is the high strength steel used in prestressing tendons. Researchers have found that there was very little increase in the upper yield stress and ultimate tensile strengths of high strength steels under dynamic loading.

The dynamic design strength is obtained by multiplying the static design strength by the appropriate dynamic increase factor DIF, which is as follows:

(a)	Concrete:	Compression	DIF = 1.19
		Diagonal tension	DIF = 1.00
		Direct shear	DIF = 1.10
		Bond	DIF = 1.00
(b)	Non-prestressed Steel Reinforcement;		
		Flexure	DIF = 1.17
		Shear	DIF = 1.00
(c)	Welded Wire Fabric:		DIF = 1.10
(d)	Prestressed Reinforcement		DIF = 1.00

6-13 Ultimate Strength of Precast Elements

The ultimate strength of non-prestressed precast members is exactly the same as cast-in-place concrete members and as such is not repeated here. For the ultimate strength of non-prestressed precast elements, see Volume IV of this manual.

6-13.1 *Ultimate Dynamic Moment Capacity of Prestressed Beams*

The ultimate dynamic moment capacity M_u of a prestressed rectangular beam (or of a flanged section where the thickness of the compression flange is greater than or equal to the depth of the equivalent retangular stress block, a) is as follows:

$$M_u = A_{ps}f_{ps}(d_p - a/2) + A_s f_{dy}(d-a/2) \quad 6\text{-}20$$

and

$$a = \frac{(A_{ps}f_{ps} + A_s f_{dy})}{.85 f'_{dc} b} \quad 6\text{-}21$$

where:

M_u = ultimate moment capacity
A_{ps} = total area of prestress reinforcement
f_{ps} = average stress in the prestressed reinforcement at ultimate load
d_p = distance from extreme compression fiber to the centroid of the prestressed reinforcement

a = depth of equivalent rectangular stress block
A_s = total area of non-prestressed tension reinforcement
f_{dy} = dynamic design strength of non-prestressed reinforcement
d = distance from extreme compression fiber to the centroid of the non-prestressed reinforcement
f'_{dc} = dynamic compressive strength of concrete
b = width of the beam for a rectangular section or width of the compression flange for a flanged section

The average stress in the prestressed reinforcement at ultimate load f_{ps}, must be determined from a trial-and-error stress-strain compatibility analysis. This may be tedious and difficult especially if the specific stress-strain curve of the steel being used is unavailable. In lieu of such a detailed analysis, the following equations may be used to obtain an appropriate value of f_{ps}:

For members with bonded prestressing tendons:

$$f_{ps} = f_{pu} \left(1-\frac{\gamma_p}{\beta_1}\left[p_p \frac{f_{pu}}{f'_{dc}} + \frac{d f_{dy}}{d_p f'_{dc}}(p-p')\right]\right) \qquad 6\text{-}22$$

and

$$p_p = A_{ps}/bd_p \qquad 6\text{-}23$$

$$p = A_s/bd \qquad 6\text{-}24$$

$$p' = A'_s/bd \qquad 6\text{-}25$$

where:

f_{pu} = specified tensile strength of prestressing tendon
γ_p = factor for type of prestressing tendon
= 0.40 for $f_{py}/f_{pu} \geq 0.80$
= 0.28 for $f_{py}/f_{pu} \geq 0.90$
f_{py} = "fictitious" yield stress of prestressing tendon corresponding to a 1 percent elongation
β_1 = 0.85 for f'_{dc} up to 4000 psi and is reduced 0.05 for each 1000 psi in excess of 4000 psi
p_p = prestressed reinforcement ratio
p = ratio of non-prestressed tension reinforcement
p' = ratio of compression reinforcement
A'_s = total area of compression reinforcement

If any compression reinforcement is taken into account when calculating f_{ps} then the distance from the extreme compression fiber to the centroid of the compression reinforcement must be less than $0.15d_p$ and

$$p_p \frac{f_{pu}}{f'_{dc}} + \frac{f_{dy}}{d_p f'_{dc}}(p-p') \geq 0.17 \qquad 6\text{-}26$$

If there is no compression reinforcement and no non-prestressed tension reinforcement, Equation 6-22 becomes:

$$f_{ps} = f_{pu}\left[1 - \frac{\gamma_p}{\beta_1} p_p\left(\frac{f_{pu}}{f'_{dc}}\right)\right] \qquad 6\text{-}27$$

For members with unbonded prestressing tendons and a span-to-depth ratio less than or equal to 35:

$$f_{ps} = f_{se} + 10{,}000 + f'_{dc}/(100 p_p) \leq f_{py} \qquad 6\text{-}28a$$

and

$$f_{ps} \leq f_{se} + 60{,}000 \qquad 6\text{-}28b$$

where:

f_{se} = effective stress in prestressed reinforcement after allowances for all prestress losses

For members with unbonded prestressing tendons and a span-to-depth ratio greater than 35:

$$f_{ps} = f_{se} + 10{,}000 + f'_{dc}/(300\, p_p) \leq f_{py} \qquad 6\text{-}29a$$

and

$$f_{ps} \leq f_{se} + 30{,}000 \qquad 6\text{-}29b$$

To insure against sudden compression failure the reinforcement ratios for a rectangular beam, or for a flanged section where the thickness of the compression flange is greater than or equal to the depth of the equivalent retangular stress block will be such that:

$$\frac{p_p f_{ps}}{f'_{dc}} + \frac{d f_{dy}}{d_p f'_{dc}}(p - p') \leq 0.36\beta_1 \qquad 6\text{-}30$$

When the thickness of the compression flange of a flanged section is less than the depth of the equivalent retangular stress block, the reinforcement ratios will be such that

$$\frac{p_{pw} f_{ps}}{f'_{dc}} + \frac{d f_{dy}}{d_p f'_{dc}}(p_w - p'_w) \leq 0.36\,\beta_1 \qquad 6\text{-}31$$

p_{pw}, p_w, p'_w = reinforcement ratios for flanged sections computed as for p_p, p and p' respectively except that b shall be the width of the web and the reinforcement area will be that required to develop the compressive strength of the web only.

6-13.2. *Diagonal Tension and Direct Shear of Prestressed Elements*

Under conventional service loads, prestressed elements remain almost entirely in compression, and hence are permitted a higher concrete shear stress than non-prestressed elements. However at ultimate loads the effect of prestress is lost and thus no increase in shear capacity is permitted. The shear capacity of a precast beam may be calculated using the equations of Volume IV of this manual. The loss of the effect of prestress also means that d is the actual distance to the prestressing tendon and is not limited to 0.8h as it is in the ACI code. It is obvious then that at the supports of an element with draped tendons, d and thus the shear capacity are greatly reduced. Draped tendons also make it difficult to properly anchor shear reinforcement at the supports, exactly where it is needed most. Thus it is recommended that only straight tendons be used for blast design.

6-14 Dynamic Analysis

The dynamic analysis of precast elements uses the procedures described in Volume III of this manual.

Since precast elements are simply supported, the resistance-deflection curve is a one-step function (see fig. 3-39a). The ultimate unit resistance for various loading conditions is presented in table 3-1. As precast structures are subject to low blast pressures, the dead load of the structures become significant, and must be taken into account.

The elastic stiffess of simply supported beams with various loading conditions is given in table 3-7. In determining the stiffness, the effect of cracking is taken into account by using an average moment of inertia I_a, as follows:

$$I_a = (I_g + I_c)/2 \qquad 6\text{-}32$$

where:

I_g = moment of inertia of the gross section
I_c = moment of inerita of the cracked section

For non-prestressed elements, the cracked moment of inertia can be determined from Volume IV. For prestressed elements the moment of inertia of the cracked section may be approximated by:

$$I_c = nA_{ps}d_p^2[1-(p_p)^{1/2}] \qquad 6\text{-}33$$

where n is the ratio of the modulus of elasticity of steel to concrete. The load-mass factors, used to convert the mass of the actual system to the equivalent mass, are given in table 3-12 For prestressed elements the load-mass factor in the elastic range is used. An average of the elastic and plastic range load-mass factors is used in the design of non-prestressed elements.

The equivalent single-degree-of-freedom system is defined in terms of its ultimate resistance r_u, equivalent elastic deflection X_E, and natural period of vibration T_N. The dynamic load is defined by its peak pressure P and duration T. The figures given in Volume III may be used to determine the

response of an element in terms of its maximum deflection X_m, and the time to reach maximum deflection t_m.

Recommended maximum deflection criteria for precast elements is as follows:

(1) For prestressed flexural members:

$\theta_{max} \leq 2°$ or $\mu_{max} \leq 1$, whichever governs

(2) For non-prestressed flexural members

$\theta_{max} \leq 2°$ or $\mu_{max} \leq 3$, whichever governs

(3) For compression members

$\mu_{max} \leq 1.3$

where θ = maximum support ratio

μ_{max} = maximum ductility ratio

6-15 Rebound

Precast elements will vibrate under dynamic loads, causing negative deflections after the maximum deflection has been reached. The negative forces associated with these negative deflections may be predicted using figure 3-268.

6-15.1 *Non-prestressed elements*

The design of non-prestressed precast element for the effects of rebound is the same as for cast-in-place members. See volume IV for a discussion of rebound effects in concrete elements.

6-15.2 *Prestressed elements*

In prestressed elements, non-prestressed reinforcement must be added to what is the compression zone during the loading phase to carry the tensile forces of the rebound phase. The rebound resistance will be determined from figure 3-268, but in no case will it be less than one-half of the resistance available to resist the blast load.

The moment capacity of a precast element in rebound is as follows:

$$M_u^- = A_s^- f_{dy}(d^- - a^-/2) \qquad 6\text{-}34$$

where:

M_u^- = ultimate moment capacity in rebound

A_s^- = total area of rebound tension reinforcement

f_{dy} = dynamic design strength of reinforcement

d^- = distance from extreme compression fiber to the centroid of the rebound reinforcement

a^- = depth of the equivalent retangular stress block

It is important to take into account the compression in the concrete due to prestressing and reduce the strength available for rebound. For a conservative design, it may be assumed that the compression in the concrete due to prestressing is the maximum permitted by the ACI code, i.e. $0.45\ f'_c$. Thus the concrete strength available for rebound is

$$0.85\ f'_{dc} - 0.45\ f'_c = 0.85f'_{dc} - 0.45f'_{dc}/DIF = 0.47f'_{dc} \qquad 6\text{-}35$$

A more detailed analysis may be performed to determine the actual concrete compression due to prestress. In either case the maximum amount of rebound reinforcement added will be

$$A_s^- \le \left[\frac{(0.85f'_{dc} - f)\beta_1}{f_{dy}}\right]\left[\frac{(87000 - nf)\ bd^-}{(87000 - nf + f_{dy})}\right] \qquad 6\text{-}36$$

where f is the compression in the concrete due to prestressing and all the other terms have been defined previously. If available concrete strength is assumed to be $0.47f'_{dc}$, equation 6-36 becomes:

$$A_s^- \le \frac{(0.47f'_{dc}\beta_1)}{f_{dy}}\ \frac{(87{,}000 - 0.378nf'_{dc})bd^-}{(87{,}000 - 0.378nf'_{dc} + f_{dy})} \qquad 6\text{-}37$$

6-16 Connections

6-16.1 *General*

One of the fundamental differences between a cast-in-place concrete structure and one consisting of precast elements is the nature of connections between members. For precast concrete structures, as in the case of steel structures, connections can be detailed to transmit gravity loads only, gravity and lateral loads, or moments in addition to these loads. In general though, connectors of precast members should be designed so that blast loads are transmitted to supporting members through simple beam action. Moment-resisting connections for blast resistant structures would have to be quite heavy and expensive because of the relatively large rotations, and hence induced stresses, permitted in blast design.

In the design of cor ··tions the capacity reduction factor ϕ, for shear and bearing stresses ··ncrete are as prescribed by ACI code, i.e. 0.85 and 0.7 respectively. No capacity reduction factor is used for moment calculations and no dynamic increase factors are used in determining the capacity of a

connector. Capacity of the connection should be at least 10 percent greater than the reaction of the member being connected to account for the brittleness of the connection. In addition the failure mechanism should be controlled by tension or bending stress of the steel, and therefore the pullout strength of the concrete and the strength of the welds should be greater than the steel strength.

The following connections are standard for use in blast design but they are not intended to exclude other connection details. Other details are possible but they must be able to transmit gravity and blast loads, rebound loads and lateral loads without inducing moments.

6-16.2 *Column-to-Foundation Connection*

The standard PCI column-to-foundation connection may be used for blast design without modification. However anchor bolts must be checked for tension due to rebound in order to prevent concrete pullout.

6-16.3 *Roof Slab-to-Girder Connection*

Figure 6-13 shows the connection detail of a roof panel (tee section) framing into a ledger beam. The bearing pads transmit gravity loads while preventing the formation of moment couples. The bent plate welded to the plate embedded in the flange of the tee transmits lateral loads but is soft enough to deform when the roof panel tries to rotate. The angle welded to the embedded plate in the web of the tee restricts the panel, through shear action, from lifting off the girder during the rebound loading. The effects of dimensional changes due to creep, shrinkage and relaxation of prestress should be considered in this type of connection.

6-16.4 *Wall Panel-to-Roof Slab Connection*

The basic concepts employed in the roof slab-to-girder connection apply to the wall panel-to-roof slab connection shown in figure 6-14. The roof panel instead of bearing on the girder, bears on a corbel cast with the tee section. The angle that transmits lateral loads has been moved from the underside of the flange to the top of the flange to facilitate field welding.

6-16.5 *Wall Panel-to-Foundation Connection*

The wall panel in figure 6-15 is attached to the foundation by means of angles welded to plates cast in both the wall panel and the foundation. It is essential to provide a method of attachment to the foundation that is capable of taking base shear in any direction, and also a method of levelling and aligning the wall panel. Non-shrinking grout is used to fill the gap between the panel and the foundation so as to transmit the loads to the foundation.

6-16.6 *Panel Splice*

Since precast structures are of the shear wall type, all horizontal blast loads are transferred by diaphram action, through wall and roof slabs to the foundations. The typical panel splice shown in figure 6-16 is used for transferring the horizontal loads between panels.

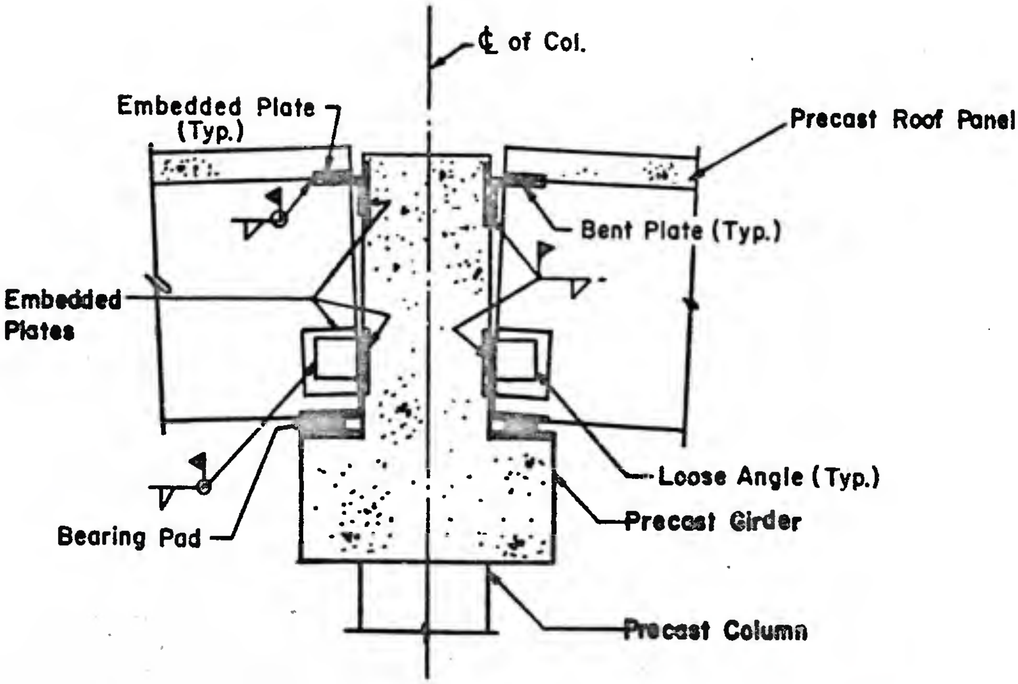


Figure 6-13 Roof slab-to-girder connection

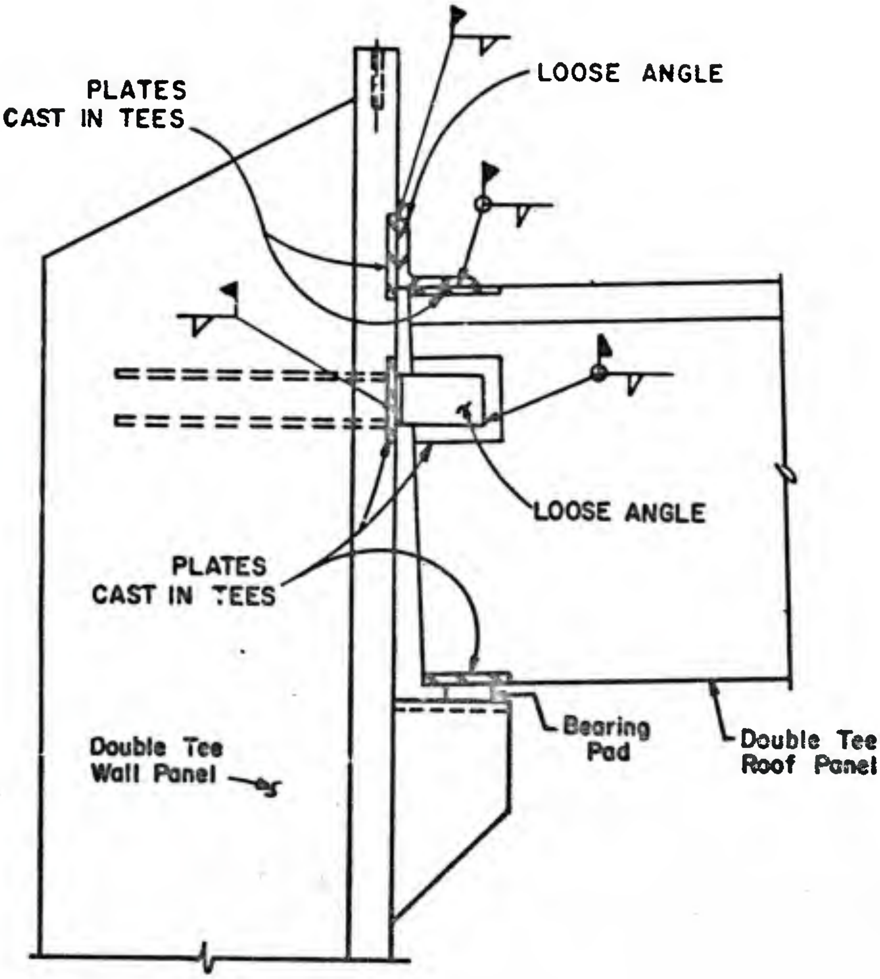


Figure 6-14 Typical wall panel-to-roof slab connection

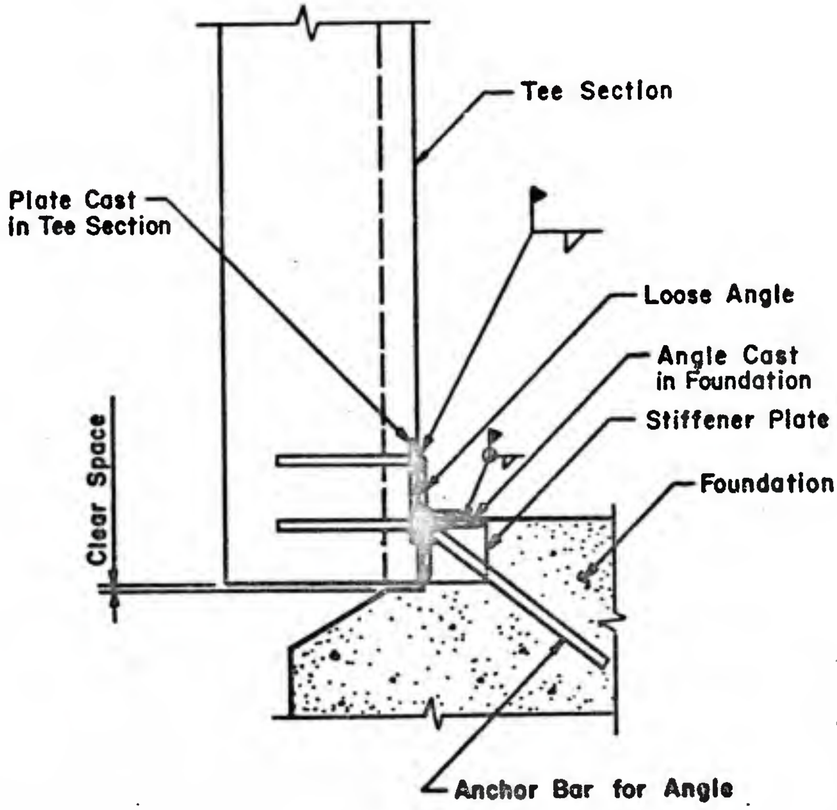


Figure 6-15 Wall panel-to-foundation connection

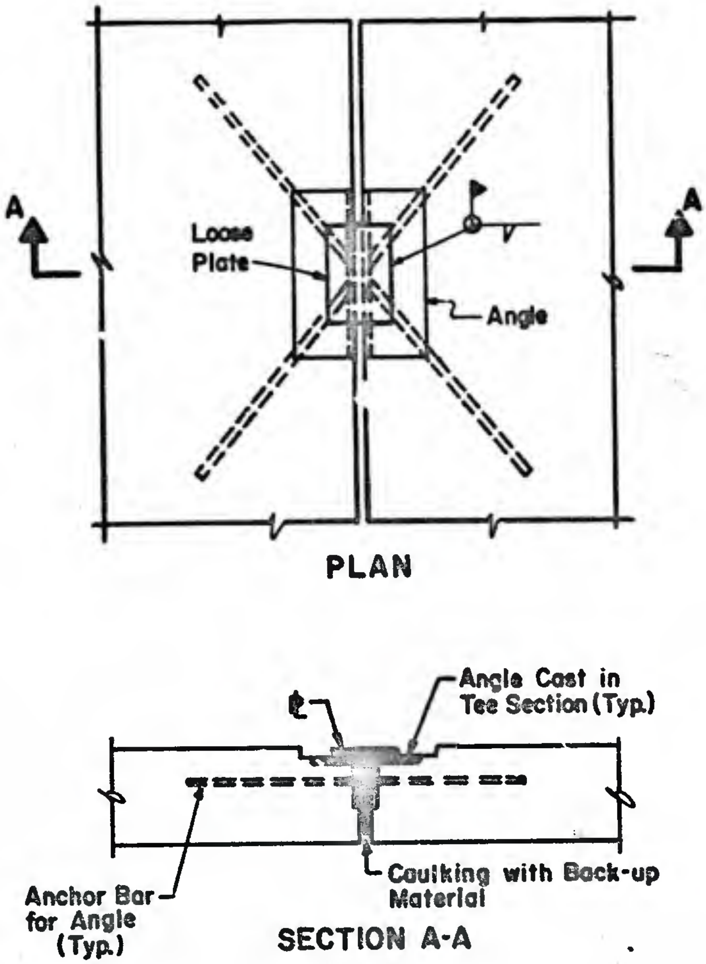


Figure 6-16 Typical panel splice

6-16.7 *Reinforcement Around Door Openings*

A standard double tee setion cannot be used around a door opening. Instead a special panel must be fabricated to satisfy the requirements for the door opening. The design of the reinforcement around the door opening and the door frame is discussed in Volume IV.

SPECIAL PROVISIONS FOR PRE-ENGINEERED BUILDING

6-17 General

Standard pre-engineered buildings are usually designed for conventional loads (live, snow, winc and/or seismic). Blast resistant pre-engineered buildings are also designed in the same manner as standard structures. However, the conventional loadings, which are used for the latter designs, are quite large to compensate for effects of blast loads. Further, as with standard buildings, pre-engineered structures, which are designed for blast, are designed elastically for the conventional loadings with the assumption that the structure will sustain plastic deformations due to the blast. The design approach will require a multi-stage process, including: preparation of general layouts and partial blast designs by the design engineer; preparation of the specifications, by the engineer including certain features as recommended herein; design of the building and preparation of shop drawings by the pre-engineered building manufacturers; and the final blast evaluation of the structure by design engineer utilizing the layouts on the previously mentioned shop drawings. At the completion of the analysis some slight modifications in building design may be necessary. However, if the following procedures are used, then the required modifications will be limited and in some cases eliminated for blast overpresures upward to 2 psi.

6-18 General Layout

The general layout of pre-engineered buildings is based on both operational and blast resistant requirements. Figure 6-17 illustrates a typical general layout of the pre-engineered building. The general requirements for structural steel, concrete, wall and roof coverings and connections are given below.

6-18.1 *Structural Steel*

In order for a pre-engineered building to sustain the required blast loading, structural steel layout must conform to the following requirements:

1. The maximum spacing between main transverse rigid frames (bay width) shall not exceed 20 feet.

2. The maximum spacing between column supports for rigid frames shall not exceed 20 feet while the overall height of frames shall be 30 feet or less.

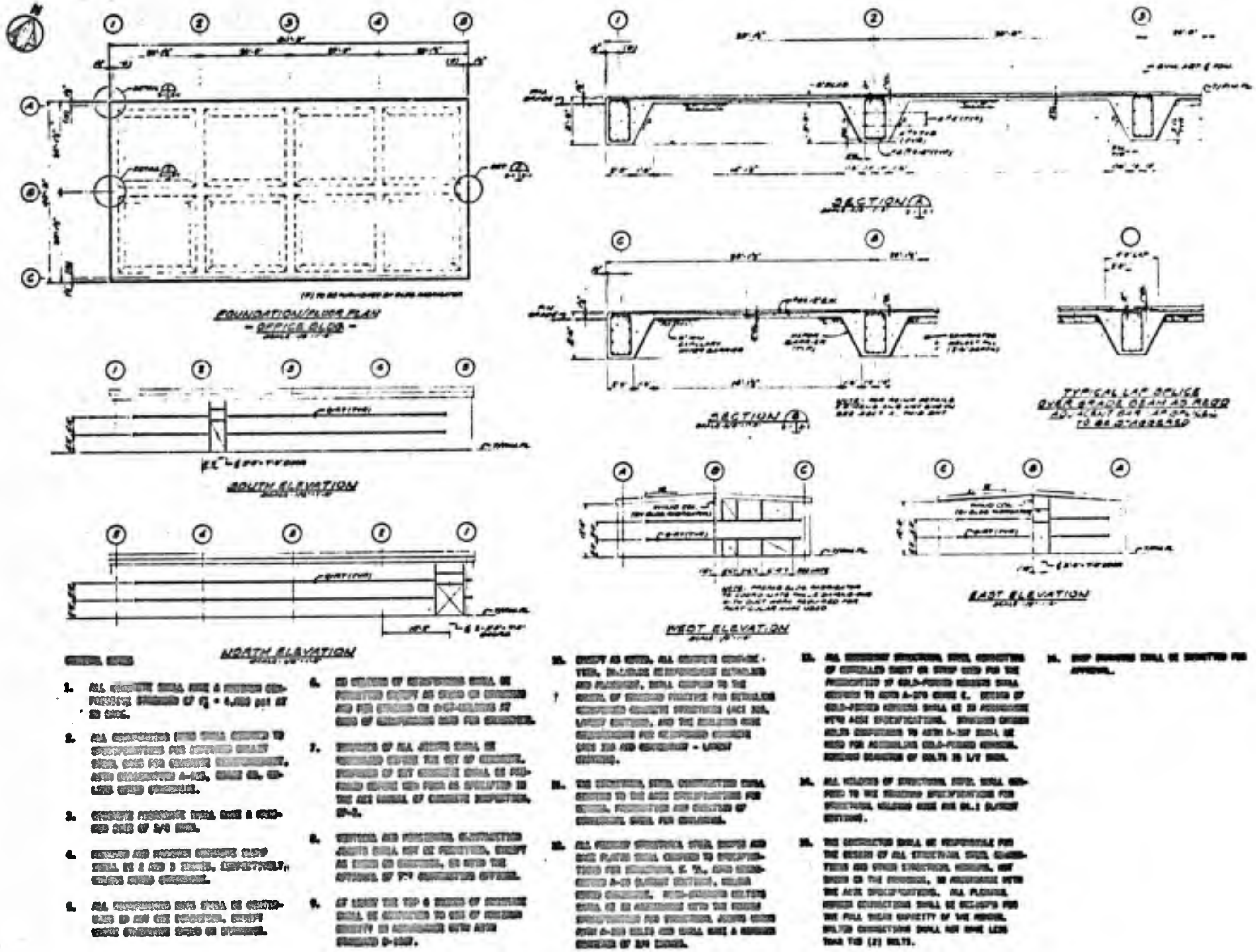


Figure 6-17 General layout of pre-engineered building

3. Slope of the roof shall not exceed four horizontal to one vertical. However, the roof slope shall be as shallow as physically possible and be in compliance with the requirements of the Metal Building Manufacturers' Association.

4. Spacing between girts shall not exceed 4 feet while the space between purlins shall not be greater than 5 feet.

5. Primary members, including frames and other main load carrying members, shall consist of hot rolled structural steel shapes. The shapes must be doubly-symmetrical an have a constant depth. They may be wide-flange sections, I-sections, structural tubes, or welded shapes built-up from hot rolled steel sheet, strips or plates. Secondary structural framing, such as girts, roof purlins, bridging, eave struts and other miscellaneous secondary framing, may consist of either hot rolled or cold-formed structural steel. All main secondary members (purlins, girts, etc.) shall be doubly-symmetrical sections of constant depth (e.g. wide flange, "I"-shaped, structural tubing).

6. Primary structural framing connections shall be either shop welded or bolted or field bolted assemblies. ASTM A 325 bolts with appropriate nuts and washers shall be used for connecting of all primary members; where as secondary members may use bolts conforming to ASTM A 307. A minimum of two bolts shall be used for each connection while bolts for primary and secondary members shall not be less than 3/4 and 1/2-inch in diameter, respectively.

7. Base plates for columns shall be rolled and set on grout bed of 1-inch minimum thickness. ASTM A 307 steel bolts shall be used to anchor all columns.

6-18.2 *Foundations*

Concrete floor and foundation slabs shall be monolithic in construction and shall be designed to transfer all horizontal and vertical loads from the pre-engineered superstructure to the foundation soil. Minimum slab thickness shall be 6 inches with edge beams thickened to meet local frost conditions.

6-18.3 *Roof and Walls*

Roof and wall coverings must meet the following requirements:

1. Roof and wall coverings shall conform to ASTM A 446, G 90, have a minimum depth of 1-1/2 inches corrugation and have a material thickness of 22 guage.

2. Conventional side laps are not usually sufficient to resist the effects of blast loads. The construction details required to strengthen those joints depend upon the type of decking employed. Volume V gives the required panel-to-panel attachments for various types of decking.

3. Insulation retainers or sub girts shall be designed to transmit all external loads (listed below) which act on the metal cover to the structural steel framing.

4. Roof and wall liners shall be a minimum of 24 gauge and shall be formed to prevent waviness, distortion or failure as a result of the impact by external loads.

6-18.4 *Connections for Roof and Wall Coverings*

The connections used in a blast resistant structure are especially critical. To ensure full development of structural steel and the roof and wall panels, connections must meet the following criteria:

1. Fasteners for connecting roof and wall coverings to structural steel supports shall be designed to support the external loads (listed below) and shall consist of either self-tapping screws, self-drilling and self-tapping screws, bolts and nuts, self-locking rivets, self-locking bolts, end welded studs, or welds. Fasteners of covering to structural steel shall be located at valleys of the covering and shall have a minimum of one fastener per valley.

2. Fasteners which do not provide positive locking such as self-tapping screws, etc. shall not be used at side laps and for fastening accessories to panels. At least one fastener for side laps shall be located in each valley and at a maximum spacing along the valley of 8 inches.

3. Self tapping screws shall not have a diameter smaller than a no. 14 screw while the minimum diameter of a self-drilling and self-tapping type shall be equal to or greater than a no. 12 screw. Automatic-welded studs shall be shouldered type and have a shank diameter of at least 3/16 inch. Fasteners for use with power actuated tools shall have a shank diameter of not less than 1/2 inch. Blind rivets shall be stainless steel type and have a minimum diameter of 1/8 inch. Rivets shall be threaded stem type if used for other than fastening trim and if of the hollow type shall have closed ends. Bolts shall not be less than 1/4 inch in diameter and will be provided with suitable nuts and washers.

4. If suction and/or rebound loads dictate, provide oversized washers with a maximum outside diameter of 2 inches or a 22 gauge thick metal strip along each valley.

6-19 Preparation of Partial Blast Analysis

A partial blast analysis of a pre-engineered building shall be performed by the design engineer. This analysis shall include the determination of the minimum size of the roof and wall panels which is included in the design specifications and the design of the building foundation and floor slab. The foundation and floor slab shall be designed monolithically and have a minimum thickness as previously stated. Slab shall be designed for a foundation load equal to either 1.3 times the yield capacity of the building roof equivalent blast load or the static roof and floor loads listed below. Quite often the

foundation below the building columns must be thickened to distribute the column loads. For the blast analysis of the building foundation and floor slab, the dynamic capacity of the soil below the foundation slab can conservatively be assumed to be equal to twice the static soil capacity. The resistance of the roof of the building can be determined in accordance with the procedures given in Volume V. The front panel of the building is designed in the same manner as the roof panel. The blast loads for determining the capacities of the roof and wall panels can be determined from Volume II.

6-20 Pre-Engineered Building Design

Design of the pre-engineered building shall be performed by the pre-engineered building manufacturer using static loads and conventional stresses.

Conventional stresses are listed in "Specification for the Design, Fabrication and Erection of Structural Steel for Buildings with Commentary. Static design loads shall be as follows:

1. Floor live loads shall be as specified in the report titled "American National Standard Building Code Requirements for Minimum Design Loads in Buildings and Other Structures" (hereafter referred to as ANSI) but not less than 150 pounds per square foot.

2. Roof live loads shall be as specified ANSI.

3. Dead loads are based on the materials of construction.

4. Wind pressure shall be as computed in accordance with ANSI for exposure "C" and a wind speed of 100 miles per hour.

5. Seismic loads will be calculated according to the Uniform Building Code for the given area. If this load is greater than the computed wind pressure, than the seismic load will be substituted for wind load in all load combinations.

6. Auxiliary and collateral loads are all design loads not listed above and include suspended ceilings, insulation, electrical systems, mechanical systems, etc.

Combinations of design loads shall include the following (a) dead loads plus live loads; (b) dead loads plus wind loads, and (c) 75 percent of the sum of dead, live and wind loads.

6-21 Blast Evaluation of the Structure

Blast evaluation of the structure utilizing the shop drawings prepared in connection with the above design shall be performed by the design engineer. A dynamic analysis which describes the magnitude and direction of the elasto-plastic stresses developed in the main frames and secondary members as a result of the blast loads, shall be performed using the methods described in Volume V. This evaluation should be made at the time of the shop drawing review stage.

6-22 Recommended Specification for Pre-Engineered Buildings

Specifications for pre-engineered buildings shall be consistent with the recommended design changes set forth in the preceding Section. These example specifications are presented using the Construction Specification Institute (CSI) format and shall contain as a minimum the following:

1. APPLICABLE PUBLICATIONS. The following publications of the issues listed below, but referred to thereafter by basic designatin only, form a part of this specification to the extent indicated by the reference thereto:

1.1 American Society of Testing and Materials (ASTM)

A 36 Structural Steel

A 307-80 Standard Specification for Carbon Steel Externally and Internally Threaded Standard Fasteners

A 325 Standard Specification for High Strength Bolts for Structural Steel Joint Including Suitable Nuts and Plain Hardened Washers

A 446 Specification for Steel Sheet, Zinc Coated (Galvanized) by the Hot-Dip Process, Physical (Structural) Quantity

A 501 Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing

A 529 Standard Specification for Structural Steel with 42,000 psi Minimum Yield Point

A 570 Standard Specification for Hot-Rolled Carbon Steel Sheet and Strip, Structural Quality

A 572 Specification of High-Strength Low-Allow Columbium-Vanadium Steels of Structural Quality

1.2 American Iron and Steel Institute (AISI)

Specification for the Design of Cold-Formed Steel Structural Members and Commentary

1.3 American National Standards Institute (ANSI)

A58.1 Minimum Design Loads for Buildings and Other Structures

B18.22.1 Plain Washers

1.4 American Institute of Steel Construction (AISC)

Specification for the Design Fabrication and Erection of Structural Steel for Buildings with Commentary

Research Council on Riveted and Bolted Structural Joints (RCRBSJ)

Specification for Structural Joints Using ASTM A 325 or A 490 Bolts

1.5 American Welding Society (AWS)

D1.1 Structural Welding Code

1.6 Metal Building Manufacturers' Association (MBMA)

Metal Buildings Systems Manual

1.7 Uniform Building Code

2. GENERAL.

2.1 This section covers the manufacture and erection of pre-engineered metal structures. The structure manufacturer shall be regularly engaged in the fabrication of metal structures.

2.2 The structure shall include the rigid framing, which are spaced at a maximum of 20 feet on center, roof and wall covering, trim, closures, and accessories as indicated on the drawings. Minor alterations in dimensions shown on the drawings will be considered in order to comply with the manufacturer's standards building system, provided that all minimum clearances indicated on the drawings are maintained. Such changes shall be submitted for review and acceptance prior to fabrication.

2.3 Drawings shall indicate extent and general assembly details of the metal roofing and sidings. Members and connections not indicated on the drawings shall be designed by the Contractor in accordance with the manufacturer's standard details. The Contractor shall comply with the dimensions, profile limitations, gauges and fabrication details shown on the drawings. Modification of details will be permitted only when approved by the Owner. Should the modifications proposed by the Contractor be accepted by the Owner, the Contractor shall be fully responsible for any re-design and re-detailing of the building construction effected.

3. DEFINITIONS.

3.1 Low Rigid Frame. The building shall be single gable type with the roof slope not to exceed one on four.

3.2 Framing.

3.2.1 Primary Structural Framing. The primary structural framing includes the main transverse frames and other primary load carrying members and their fasteners.

3.2.2 Secondary Structural Framing. The secondary structural framing includes the girts, roof purlins, bridging, eave struts, and other miscellaneous secondary framing members and their fasteners.

3.2.3 Roof and Wall Covering. The roof and wall covering includes the exterior ribbed metal panel having a minimum depth of one and one-half inches, neoprene closure, fasteners and sealant.

3.3 Building Geometry.

3.3.1 Roof Slope. The roof of the building shall have a maximum slope not to exceed one on four.

3.3.2 Bay Spacing. The bay spacing shall not exceed 20 feet.

3.4 Column Shape. Main frame columns shall be doubly symmetrical members of constant depth; tapered columns will not be permitted.

3.5 Calculations. The Contractor shall submit for reivew complete design calculations for all work, sealed by a registered professional engineer.

4. STRUCTURAL DESIGN.

4.1 Structural Analysis. The structural analysis of the primary and secondary framing and covering shall be based on linear elastic behavior and shall accurately reflect the final configuration of the structure and all tributary design loadings.

4.2 Basic Design Loads.

4.2.1 Roof Live Load. Shall be applied to the horizontal roof projection. Roof live loads shall be:

0 to 200 square feet tributary area - 20 psf

200 to 600 square feet tributary area - linear variation 20 psf to 12 psf

over 600 square feet tributary area - 12 psf

4.2.2 Wind Pressure. Wind design loads shall be computed in accordance with ANSI A58.1 for exposure "C" and a basic wind speed of 100 miles per hour.

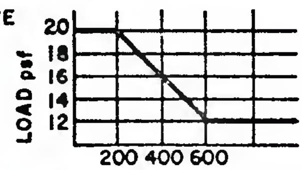
4.2.2.1 Typical Wind Loading. As shown on drawings (fig. 6-18).

4.2.2.2 Wind Loading at Building Corners. As shown on the drawings (fig. 6-18).

4.2.2.3 Wind Loading on Girts. As shown on drawings (fig. 6-18).

1. FLOOR LIVE LOAD: 150 psf

2. ROOF LIVE LOAD



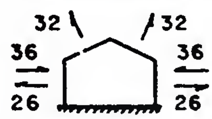
3. ROOF SNOW LOAD: 30 psf

4. DEAD LOAD: AS PER MATERIALS USED

5. WIND LOADS: WIND PARALLEL OR PERPENDICULAR TO ROOF RIDGE (BASED ON 100 mph WIND)

A. WINDWARD/LEEWARD AND ROOF PRESSURES (psf)

a. WHEN TRIBUTARY SUPPORT AREAS > 200 ft^2

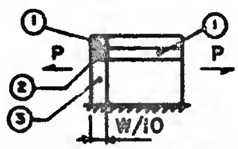


FOR DESIGN OF MAIN FRAMES AND OTHER INDIVIDUAL MEMBERS, USE VALUES SHOWN.

b. WHEN TRIBUTARY SUPPORT AREA ≤ 200 ft^2

FOR DESIGN OF MAIN FRAMES, USE VALUES SHOWN FOR TRIBUTARY AREA > 200 ft^2. FOR DESIGN OF OTHER INDIVIDUAL MEMBERS, USE VALUES SHOWN

B. SIDEWALL PRESSURES (psf)



FOR DESIGN OF INDIVUAL MEMBERS:
WHEN TRIBUTARY SUPPORTED > 200 ft^2, USE P = 32
WHEN TRIBUTARY SUPPORTED ≤ 200 ft^2, USE P = 43

C. SIDESWAY PRESSURES (psf)

FOR DESIGN OF MAIN FRAMES FOR TRIBUTARY SUPPORTED AREAS, TA, LESS THAN, EQUAL TO, OR GREATER THAN, 200 ft^2.

D. LOCAL PRESSURES - SEE FIGURES IN 'B' AND 'C' FOR REGIONS OF APPLICATION

REGION 1: 50 psf SUCTION
REGION 2: 105 psf SUCTION
REGION 3: 42 psf SUCTION

FOR DESIGN OF DECKING CONNECTIONS AT REGIONS CITED

"W" IS LEAST WIDTH OF ENCLOSED AREA

NOTE: FIGURES DEPICT WIND PERPENDICULAR TO RIDGE. FOR WIND PARALLEL TO RIDGE, USE SAME VALUES.

6. LOADING COMBINATIONS:

A. D
B. D + L
C. D + W
D. 0.75 (D + L + W)

WHERE
D = DEAD LOAD
L = LIVE LOAD
W = WIND LOAD

Figure 6-18 Recommended pre-engineered building design loads

4.2.2.4 Wind Loading on Purlins and Roof Tributary Areas. As shown on drawings (fig. 6-18).

4.2.2.5 Wind Loading for Design of Overall Structure. As shown on drawings (fig. 6-18).

4.2.3 Auxiliary and Collateral Design Loads. Auxiliary and collateral design loads are those loads other than the basic design live, dead, and wind loads; which the building shall safely withstand, such as ceilings, insulation, electrical, mechanical, and plumbing systems, and building equipment and supports.

4.3 Application of Design Loads.

4.3.1 Roof Live Load and Dead Load. The roof live load (L), and dead load (D), shall be considered as a uniformly distributed loading acting vertically on the horizontal projection of the roof.

4.3.2 Snow Loads. appl. ation of 30 psf due to snow loads.

4.3.3 Wind Loads (W). Application of forces due to wind shall conform to the latest ANSI A58.1

4.3.4 Combination of Loads. The following combinations of loads shall be considered in the design of all members of the structure:

D + L
D + W
.75 (D + L + W)

4.4 Deflection Limitations.

4.4.1 Structural Framing. The primary and secondary framing members shall be so proportioned that their maximum calculated roof live load deflection does not exceed 1/120 of the span.

5. STRUCTURAL FRAMING.

5.1 General

5.1.1 All hot rolled structural shapes and structural tubing shall have a minimum yield point of 36,000 psi in conformance with ASTM A 36 or A 501. All hot rolled steel plate, strip and sheet used in the fabrication of welded assemblies shall conform to the requirements of ASTM A 529, A 572, Grade 42 or A 570 Grade "E" as applicable. All hot rolled sheet and strip used in the fabrication of cold-formed members shall conform to the requirments of ASTM A 570, Grade"E" having a minimum yield strenth of 50,000 psi. Design of cold-formed members shall be in accordance with the AISI specifications.

5.1.2 The minimum thickness of framing members shall be:

Cold-formed secondary framing members	- 18 gauge
Pipe or tube columns	- 12 gauge
Webs of welded built-up members	- 1/8 inch
Flanges of welded built-up members	- 1/4 inch
Bracing rods	- 1/4 inch

5.1.3 All framing members shall be fabricated for bolted field assembly. Bolt holes shall be punched or drilled only. No burning-in of holes will be allowed. The faying surfaces of all bolted connections shall be smooth and free from burrs or distortions. Provide washers under head and nut of all bolts. Provide beveled washers to match sloping surfaces as required. Bolts shall be of type specified below. Members shall be straight and dimensionally accurate.

5.1.4 All welded connections shall be in conformance with the STRUCTURAL WELDING CODE D1.1 of the American Welding Society. The flange-to web welds shall be one side continuous submerged arc fillet welds. Other welds shall be by the shielded arc process.

5.2 Primary Structural Framing.

5.2.1 The primary members shall be constructed of doubly-symmetrical, hot rolled structural steel shapes or doubly-symmetrical built-up members of constant depth, welded from hot rolled steel sheet, strip or plates.

5.2.2 Compression flanges shall be laterally braced to withstand any combination of loading.

5.2.3 Bracing system shall be provided to adequately transmit all lateral forces on the building to the foundation.

5.2.4 All bolt connections of primary structural framing shall be made using high-strength zinc-plated (0.0003 bronze zinc plated) bolts, nuts, and washers conforming to ASTM A 325. Bolted connections shall have not less than two bolts. Bolts shall not be less than 3/4 inch diameter. Shop welds and field bolting are preferred. All field welds will require prior approval of the Owner. Installation of fasteners shall be by the turn-of-nut or load-indicating washer method in accordance with the specifications for structural joints of the Research Council on Rivieted and Bolted Structural Joints.

5.3 Secondary members may be constructed of either hot rolled or cold-formed steel. Purlins and girts shall be doubly symmetrical sections of constant depth and they may be built-up, cold-formed or hot rolled structural shapes.

5.3.1 Maximum spacing of roof purlins and wall girts shall not exceed 5 feet.

5.3.2 Compression flanges of purlins and girts shall be laterally braced to withstand any combination of loading.

5.3.3 Supporting lugs shall be used to connect the purlins and girts to the primary framing. The lugs shall be designed to restrain the light gauge sections from tipping or warping at their supports. Each member shall be connected to each lug by a minimum of two fasteners.

5.3.4 Vertical wall members not subjected to axial load, e.g. vertical members at door openings, shall be constant depth sections. They may consist of hot rolled or cold-form steel. They shall be either built-up, cold-formed or hot rolled "C" or "I" shapes.

5.3.5 Fasteners for all secondary framing shall be a minimum of 1/2 inch diameter (0.003 zinc plated) bolts conforming to ASTM A 307. The fasteners shall be tightened to SNUG TIGHT condition. Plain washers shall conform to ANSI standard B18.22.1.

6. ANCHORAGE.

6.1 Anchorage. The building anchor bolts for both primary and secondary columns shall conform to ASTM A 307 steel and shall be designed to resist the column reactions produced by the specified design loading. The quantity, size and location of anchor bolts shall be specified and furnished by the building manufacturer. A minimum of two anchor bolts shall be used with each column.

6.2 Column Base Plates. Base plates for columns shall conform to ASTM A 36 and shall be set on a grout bed of 1 inch minimum thickness.

7. ROOF AND WALL COVERING.

7.1 Roof and wall panels shall conform to zinc-coated steel, ASTM A 446, G 90 coating designation. Minimum depth of each panel corrugation shall be 1-1/2 inches and shall have a minimum material thickness of 22 gauge. The minimum yield strength of panel material shall be 33,000 psi. Wall panels shall be applied with the longitudinal configurations in the vertical position. Roof panels shall be applied with the longitudinal configuration in direction of the roof slope.

7.1.1 Structural properties of roof and wall panels shall be equal to or greater than the following:

Surface	Section Modulus (in^3/ft)	
	Roof	Wall
Outer Face in Compression		
Outer Face in Tension		

7.1.2 Side and End Laps. Side laps of roof and wall panels shall be fastened as shown on drawings. End laps, if required shall occur at structural steel supports and have a minimum length of 12 inches.

7.2 Insulation.

7.2.1 Semi-rigid insulation for the preformed roofing and siding shall be supplied and installed by the preformed roofing and siding manufacturer.

7.2.2 Insulation Retainers. Insulation retainers or sub girts shall be designed to transmit all external loads (wind, snow and live loads) acting on the metal panels to the structural steel framing. The retainers shall be capable of transmitting both the direct and suction loads.

7.3 Wall and Roof Liners. Wall and roof liners shall be a minimum of 24 gauge. All liners shall be formed or patterned to prevent waviness, distortion or failure as a result of the impact by external loads.

7.4 Fasteners. Fasteners for roof and wall panels shall be zinc-coated steel or corrosion-resisting steel. Exposed fasteners shall be gasketed or have gasketed washers of a material compatible with the covering to waterproof the fastener penetration. Gasketed portion of fasteners or washers shall be neoprene or other elastomeric material approximately 1/8 inch thick.

7.4.1 Type of Fasteners. Fasteners for connecting roof or wall panels to structural steel supports shall consist of self-tapping screws, self-drilling and self-tapping screws, bolts, end welded studs, and welds. Fasteners for panels which connect to structural supports shall be located in each valley of the panel and with a minimum of one fastener per valley while at end laps and plain ends, a minimum of two fasteners shall be used per valley. Fasteners shall not be located at panel crowns.

7.4.2 Fasteners which do not provide positive locking such as self-tapping screws or self-drilling and self-tapping screws shall not be used at side laps of panels and for fastening accessories to panels. Fasteners for side laps shall be located in each valley of the overlap and positioned a maximum of 8 inches on center.

7.4.3 Screws shall be not less than no. 14 diameter if self-tapping type and not less than no.12 diameter if self-drilling and self-tapping type.

7.4.4 Automatic end-welded studs shall be shouldered type with a shank diameter of not less than 3/16 inch with cap and nut for holding the covering against the shoulder.

7.4.5 Fasteners for use with power actuated tools shall have a shank diameter of not less than 1/2 inch. Fasteners for securing wall panels shall have threaded studs for attaching approved nuts or caps.

7.4.6 Blind rivets shall be stainless steel with 1/8 inch nominal diameter shank. Rivets shall be threaded stem type if used for other than fastening of trim. Rivets with hollow stems shall have closed ends.

7.4.7 Bolts shall not be less than 1/4 inch diameter, shoulders or plain shank as required with proper nuts.

7.4.8 Provide overside washers with an outside diameter of 1 inch at each fastener or a 22 gauge thick metal strip along each valley of the panel to negate pull-out of the panel around the fasteners.

SUPPRESSIVE SHIELDING

6-23 General

This manual presents methods for the design and construction of conventional reinforced concrete and steel protective facilities which provide adequate safety for hazardous operations such as munitions loading, maintenance, renovation, or demilitarization. Such safety considerations include the utilization of conventional protective barriers, total containment construction, or the use of separation distances or isolation of the specific operation from other parts of the facility using appropriate quantity distance specifications. However, an alternative available to the designer of these facilities is the use of suppressive shielding as outlined in HNDM 1110-1-2, "Suppressive Shields -- Structural Design and Analysis Handbook," 18 November 1977.

A suppressive shield is a vented steel enclosure which controls or confines the hazardous blast, fragment, and flame effects of detonations. Suppressive shielding may provide cost or safety effective alternatives to conventional facilities, depending upon the hazardous situation under study. HNDM 1110-1-2 presents procedures for design, analysis, quality control, and economic analysis of suppressive shields. In this section, a brief review of these procedures is presented. The reader should refer to HNDM 1110-1-2 for details necessary for design.

6-24 Application

Facility operations such as munitions loading, maintenance, modification, renovation, or demilitarization must be analyzed to determine which operations involve potentially catastrophic (CAT I or II, MIL STD 882A) hazards in the event of an inadvertent ignition or detonation. Where the hazard analysis shows such a potential, the facility design must provide adequate safety for those operations. The alternatives presented to the designer of the facility are varied and may include the utilization of conventional protective barricades with appropriate separation distances, reinforced concrete or steel structures, suppressive shields, or isolation of a particular operation from the rest of the facility by the appropriate quantity-distance. The decision as to which alternative system to use is based primarily on economic factors, provided all safety considerations are equal.

The facility, availability of real estate, and equipment costs to include maintenance, operation, useful life, replacement, and modification or renovation must be analyzed for each alternative method of protection. Costs will be estimated and compared over the facility life to determine the most economical mode of protection.

A major factor which is paramount in the determination of which form of protection to use is the requirement for approval of the facility by the Department of Defense Explosives Safety Board. If the designer can, based on economic factors, adapt suppressive shields in the design and support the adaptation with proven accepted analytical techniques, he should begin development of a facility concept which employs suppressive shields using those shields which have been safety approved.

6-24.1 *Safety Approved Suppressive Shields*

There are eight suppressive shield design groups that have been developed to various stages of definition. These shield groups are summarized in table 6-4 and illustrated schematically in figure 6-19. Of the design groups illustrated, five had been safety approved by the Department of Defense Explosive Safety Board in 1977.

The five suppressive shield group designs approved by the DoD Explosive Safety Board (Groups 3, 4, 5, 6, and 81 mm) have been designed to meet the requirements for most applications to ammunition load, assembly, pack (LAP) in the Munitions Production Base Modernization and Expansion Program. However, specific shield requirements will vary with other applications and, even with LAP applications, design details will vary from plant to plant and between munitions or different operations on the line. It will, therefore, frequently be necessary to modify the approved shields to adapt them to the operation under consideration.

Chapter II and Appendix A of HNDM 1110-1-2 describes the safety approved shield group designs, provides guidance concerning acceptable modifications, recommends procedures for securing safety approval of new shield designs, and provides summary information on overall dimensions of the shield structure, charge capacity, rated overpressure, fragment stopping wall thickness, and type of construction of the five approved basic shield groups.

6-24.2 *New Shield Design*

In exceptional cases where a safety approved shield cannot be made to fit a desired application, a new shield can be designed. The guidance needed to design a new shield and the procedures for obtaining the safety approval for the new design are outlined in HNDM 1110-1-2.

6-24.2.1 *Hazardous Environments.* Considering that the hazardous environments normally associated with suppressive shielding involves explosives and/or explosive ordnance, Chapter III of HNDM 1110-1-2 presents information relative to internal and external air blast, fragmentation, and fireball phenomena. This information can be used in support of blast and fragment methods in this

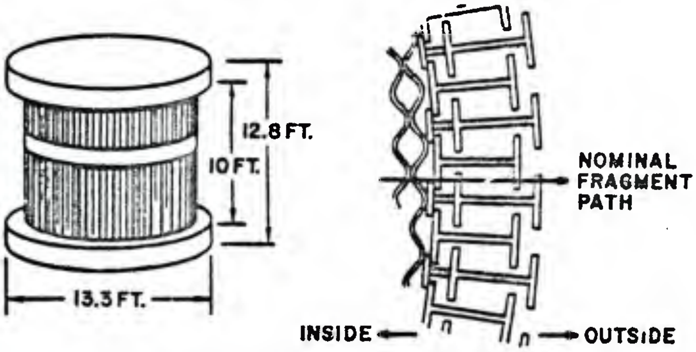
Table 6-4 Summary of Suppressive Shield Groups

SHIELD GROUP	HAZARD PARAMETER		REPRESENTATIVE APPLICATIONS	LEVEL OF PROTECTION*
	BLAST	FRAGMENTATION		
1	High	Severe	Porcupine Melter (2000 lb) plus two pour units 250 lb each	Reduce blast pressure at intraline distance** by 50 percent
2	High	Severe	HE Bulk (750 lb) Minute melter	Reduce blast pressure at intraline distance** by 50 percent
3	High	Moderate	HE bulk (37 lb) Detonators, fuses	Category I hazard*** 6.2 feet from shield
4	Medium	Severe	HE bulk (9 lb) Processing rounds	Category I hazard*** at 19 feet from shield
5	Low	Light	30 lb Illuminant Igniter slurry mixing HE processing (1.84 lb)	Category I hazard*** at 3.7 feet from shield
6	Very High	Light	Laboratory, handling, and transportation	Category I hazard*** at 1 foot from shield
7	Medium	Moderate	Flame/fireball attenuation	Category I hazard*** at 5 feet from shield
81 mm	High	Moderate	81 mm mortar drill-and-face and/or cast-finishing operation	Category I hazard*** at 3 feet from shield

* All shield groups contain all fragments

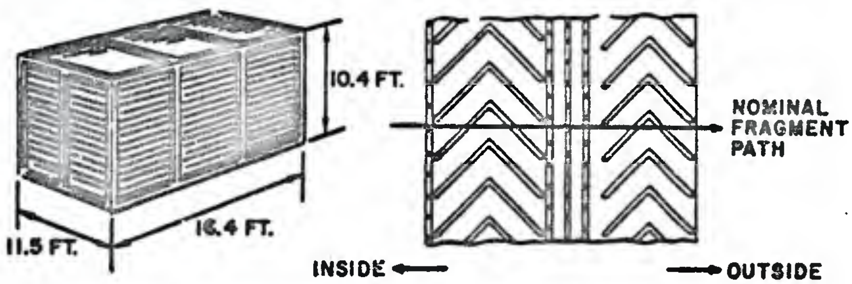
** Unbarricaded intraline distance as defined by Table 17-12 in the Safety Manual, AMCR 385-100

*** Category I hazard (2.3 psi level) as defined by MIL STD 398

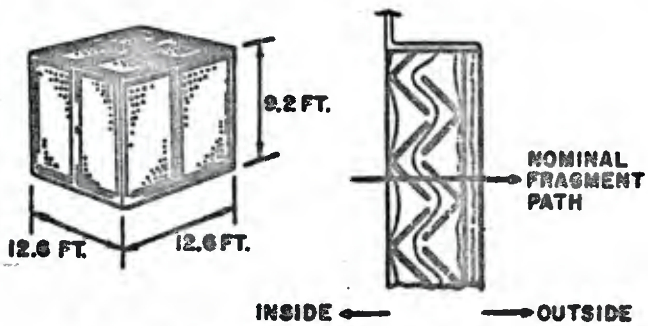


SUPPRESSIVE SHIELD GROUP 3

(GROUPS 1 & 2 ARE SIMILAR, BUT MUCH LARGER, AND HAVE THREE EXTERNAL RINGS)

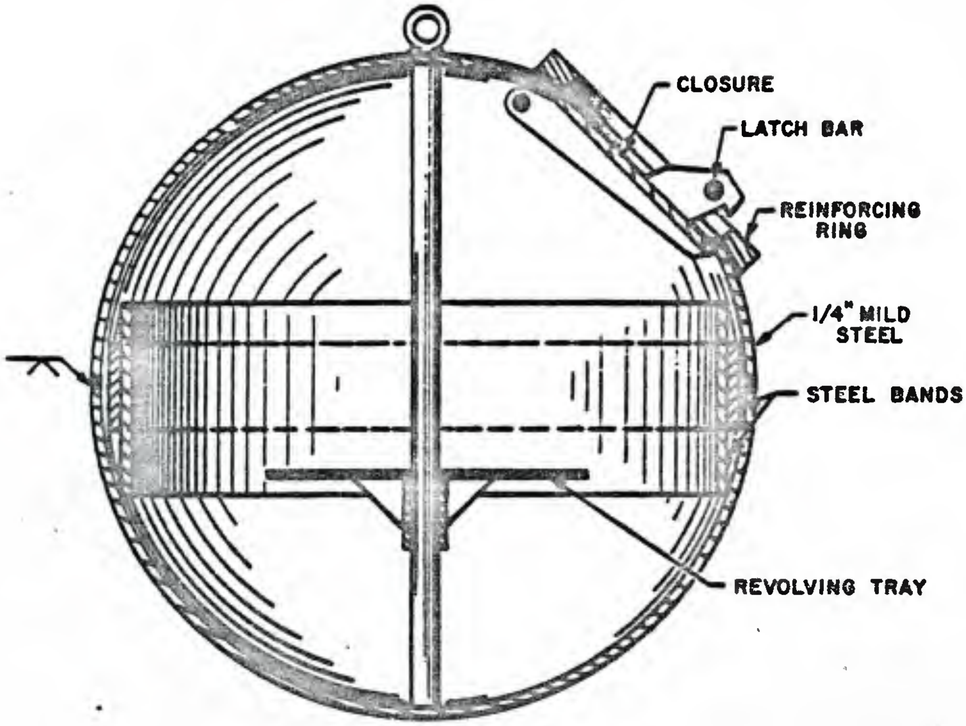


SUPPRESSIVE SHIELD GROUP 4

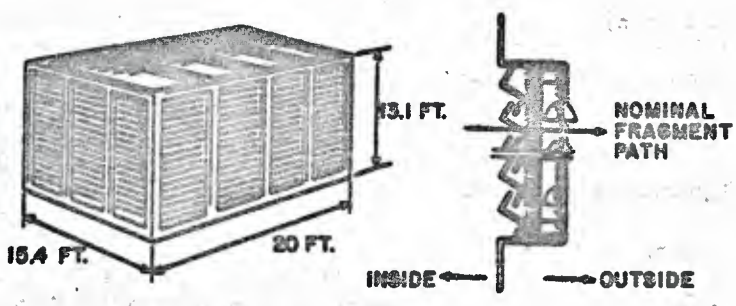


SUPPRESSIVE SHIELD GROUP 5

Figure 6-19a General configuration of suppressive shield groups



SUPPRESSIVE SHIELD GROUP 6



SUPPRESSIVE SHIELD GROUP 81mm

Figure 6-19b General configuration of suppressive shield groups (continued)

manual to determine venting requirements, air blast loads on the structure, and protection required to defeat fragments. Some graphs and prediction methods in Volume II of this manual are taken directly from the suppressive shields manual.

6-24.2.2 *Structural Behavior.* Suppressive shields can be subjected to large, high pressure loads applied very rapidly. The allowance of inelastic behavior of the shield material structural elements enables much more efficient use of the structural material and does not impair the function of the shield provided, of course, that the inelastic behavior is maintained within acceptable limits.

The structural materials of primary interest in suppressive shielding are steel and reinforced concrete. Chapter IV of HNDM 1110-1-2 discusses the behavior and properties of these structural materials under static and dynamic loading. Additionally, ductility ratios as they apply to suppressive shield application are covered. Some of the information provided will duplicate material in volumes of this manual. In case of a conflict, this manual takes precedence.

6-24.2.3 *Structural Design and Analysis.* Chapter V of HNDM 1110-1-2 describes techniques which are sufficiently accurate for preliminary designs in all cases, and in most cases, adequate for final designs. These methods deal primarily with the dynamic loadings imposed by internal explosions. The design methods supplement material presented in volumes of this manual. Again, in case of conflict, this manual takes precedence.

6-24.2.4 *Structural Details.* Each suppressive shield used for ammunition manufacturing and other hazardous operations will have specific requirements for utility penetrations, and doors for personnel, equipment, and products. Guidance on the provision of acceptable structural details such as these is presented in Chapter VI of HNDM 1110-1-2 along with information on structural details which have been successfully proof-tested.

6-24.2.5 *Economic Analysis.* The design of a facility entails the need to ascertain the most cost effective configuration from among a set of workable design alternatives. All will be designed to provide the desired level of reliability and safety, and the selection of one over another will be based primarily on dollar costs. The economic analysis of alternative facility design is a complex process unique to each facility. Chapter VII of HNDM 1110-1-2 illustrates the many factors that must be considered.

6-24.2.6 *Assuring Structural Quality.* In the design of suppressive shields, specifications for the quality of the basic material is paramount. The strength of welds and concrete components are also determining factors in the overall strength of the structure. Chapter VIII of HNDM 1110-1-2 provides the guidance which outlines a quality assurance program for suppressive shield design packages.

Included in Appendix A of HNDM 1110-1-2 is a detailed description of the safety approved suppressive shields and guidance concerning acceptable modifications. Copies of the fabrication drawings for each approved shield design are included along with direction for ordering full-size copies. Appendix B of that manual includes response charts for use in preliminary

design. The charts are based on the combined short duration shock load and infinite duration quasi-static load, along with an undamped elastic-plastic responding structure.

6-25 Design Criteria

Design criteria for use of suppressive shields, or suppressive shielding panels, are very dependent on specific applications in protective structures. These criteria may include complete suppression of fragmentation effects, both primary and secondary; attenuation of blast overpressures and impulses to specified levels of specific distances from the shield or shield panels; attenuation of fireball radiation; or even essentially complete suppression of all of these effects.

Suppressive shields may or may not present reasonable or cost effective solutions to specific design problems in protective structures. Generally, they have appeared attractive when fragment hazards are severe and when potentially explosive sources are rather concentrated. The safety-approved shields protect against effects as limited as small trays of detonators, and as severe as a large melt kettle in a HE melt-pour operation containing several thousand pounds of explosive. The designer should consider their use, and use the methods presented in HNDM 1110-1-2 to evaluate their efficacy, compared to other types of protective structures discussed in this manual.

No general design criteria can be given here because the criteria for different operations or plants, and available real estate, differ too widely. In each specific protection design contract, the AE should be provided with quite detailed design criteria, in addition to general regulations which fix safety criteria such as AMCR 385-100. Both the specific and more general criteria must be evaluated when deciding whether or not suppressive shields will be useful in the facility design.

6-26 Design Procedures

6-26.1 *Space Requirements*

Once the operation requiring suppressive shields has been identified, consideration must be given to the size and shape of the equipment needed to perform the operation and the work space required inside the shield. These factors necessarily provide the designer with an estimate of the size and shape of the shield required. Additionally, space available on the line or in the building will place limitations on the overall shield base dimensions and height.

6-26.2 *Charge Parameters*

A principal factor in the selection of a shield which will govern the shield requirements is the establishment of the charge parameters for any specific application. The charge parameters are: charge weight (W), shape, confinement, and composition; ratio of charge weight to shield internal volume (W/V); and scaled distance (Z) from the charge to the nearest wall or roof of the shield. ($Z = R/W^{1/3}$), where R is the distance from the center of the charge to the nearest wall or roof in feet and w is the charge weight in pounds. These parameters for approved shield groups are summarized in table 6-5. New shield designs can be developed for individual needs.

Table 6-5 Charge Parameters for Safety Approved Shields

MINIMUM Z (ft/lb $^{1/3}$)			
SHIELD GROUP	WALL	ROOF	MAXIMUM W/V (lb/ft^3)
3	1.63	1.45	0.04157
4	2.23	2.19	0.00762
5	4.14	6.79	0.00215
6A	1.01	N/A	0.22970
6B	1.22	N/A	0.13200
Prototype 81 mm*	3.62	3.21	0.00340
Milan 81 mm*	4.23	3.75	0.00280

* See Figure 6-19

6-26.3 *Fragment Parameters*

Another key factor in the procedure a designer follows in the selection of an approved design or the design of a new concept is the suppression of primary and secondary fragments generated by the detonation of explosives or munitions. Much of the material in HNDM 1110-1-2 for fragment perforation of spaced plates has been adapted to Volume V of this manual.

6-26.4 *Structural Details*

Suppressive shields used for ammunition manufacturing and other hazardous operations require provisions for gaining access to the operation being protected. Personnel must be able to enter the shield to acomplish routine and emergency maintenance and clean-up and other essential operations. An opening of sufficient size must be provided to enable the installation or removal of equipment in realistically large subassemblies. Openings for conveyors and chutes must also be provided and properly configured to prevent excessive pressure and fragments from escaping. Provisions must be made to provide all utilities and satisfy all environmental conditioning needs which may be essential to the operations inside the shield.

Utility penetrations, ventilating and air-conditioning ducts, and vacuum lines must not diminish the overall protective capability of the shield. They must not alter the basic mode of structural failure of the suppressive shield and should be small compared to the general size of the shield.

Operations that produce explosive dust may require the use of liners both inside and outside the shield to prevent the accumulation of dust within shield panels. With configurations such as the Group 5 shield, which is primarily designed for use with propellants or pyrotechnic materials, liners must not inhibit the venting characteristics of the shield.

Utility lines passing through suppressive shields are vulnerable to both air blast and fragment hazards. The air blast could push unprotected utility penetrations through the walls of the shield and create secondary fragments. Fragments from an accidental explosion could perforate the thin walls of an unprotected utility pipe and escape from the shield. To eliminate the threat of air blast and fragments, a protective box is used to cover the area where the utility lines pass through the shield wall. The box is configured to rest on the inside surface of the shield and is welded to the shield. The size of the wall penetrations is limited to that required for the utilities. Each pipe is bent at a right angle inside the shield within the protective box. The penetrations of the shield wall are reinforced with a sleeve or box section welded to the shield panel through which the utility line passes. The penetration box is designed to maintain the structural integrity of the shield area penetrated. A typical protective box design is shown in figure 6-20. The cover plate thickness is selected to stop the worst case fragment.

Typical penetrations for approved safety design suppressive shields are illustrated in figures 6-21 and 6-22, and a vacuum line penetration is illustrated in figure 6-23.

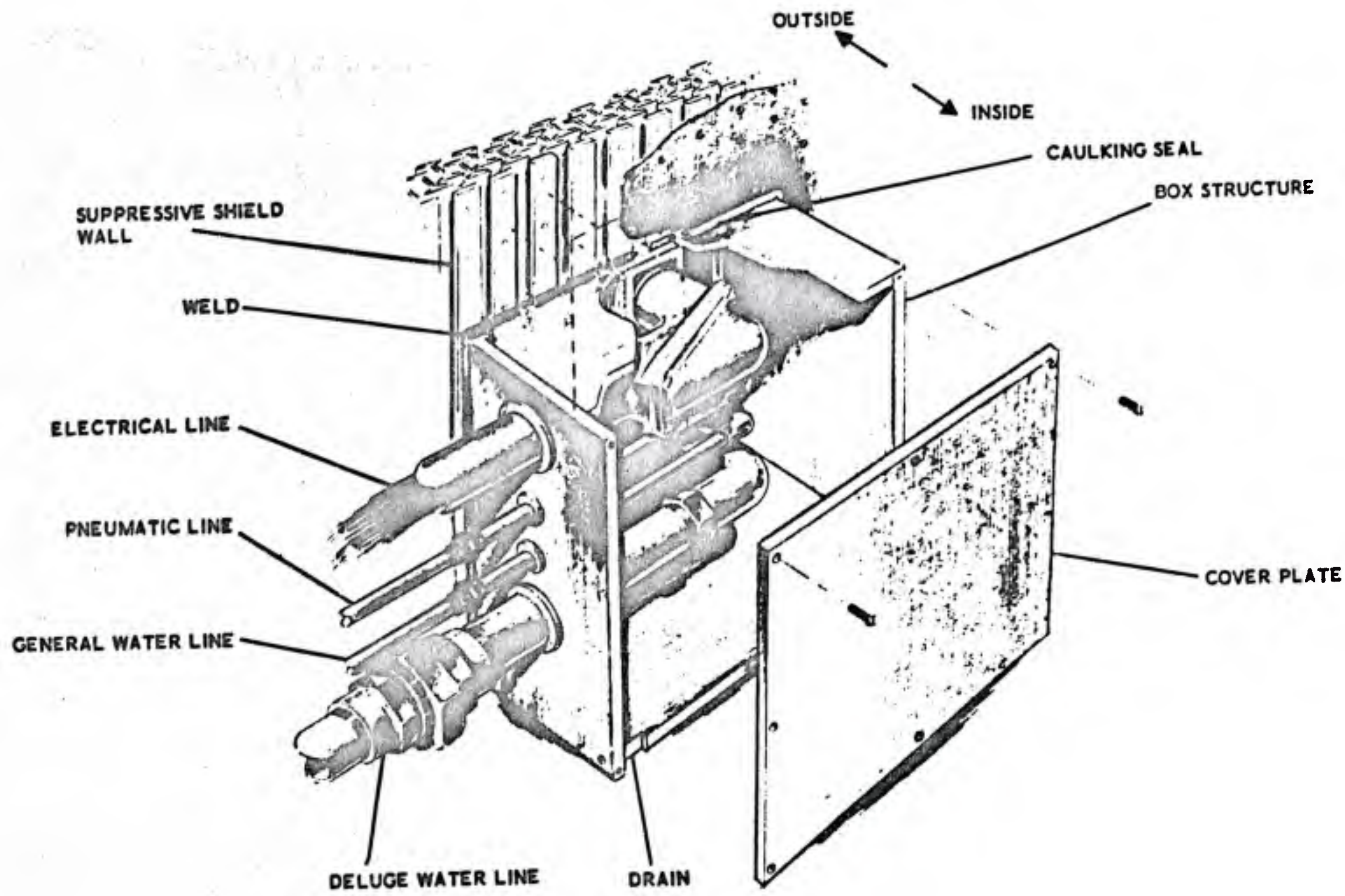


Figure 6-20 Typical utility penetration

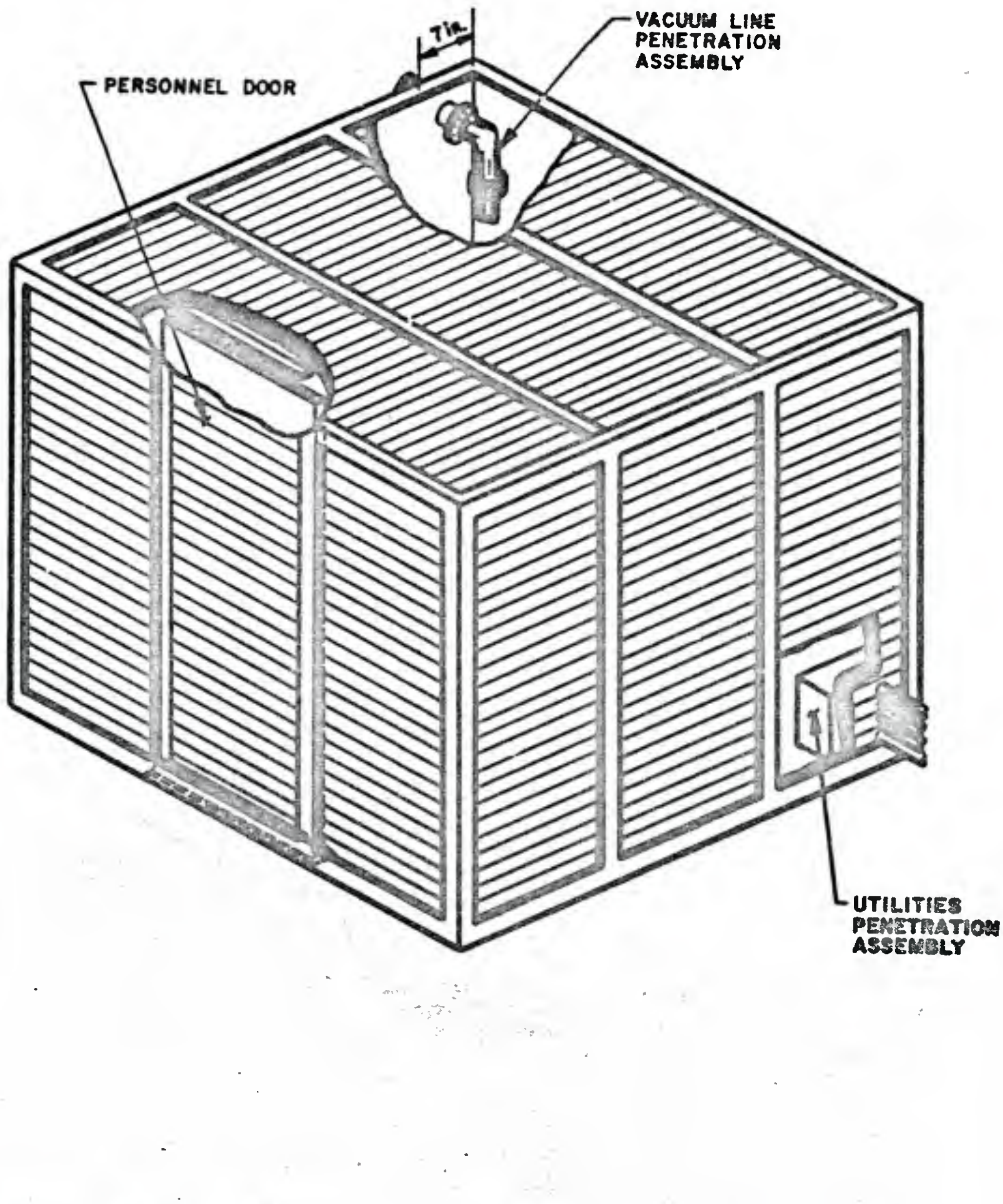


Figure 6-21 Typical location of utility penetration in shield groups 4, 5, and 81 mm

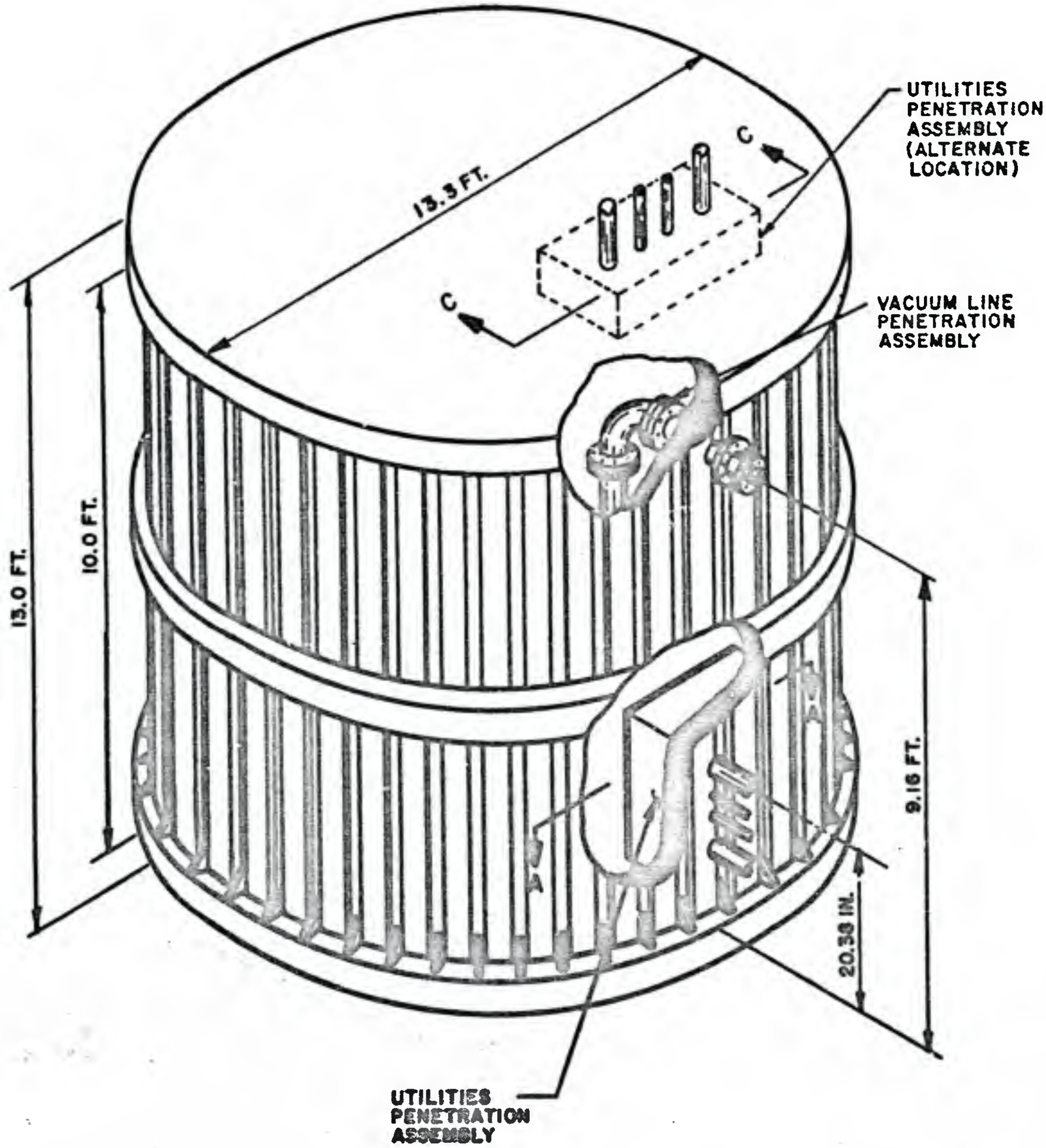


Figure 6-22 Typical location of utility penetrations in shield group 3

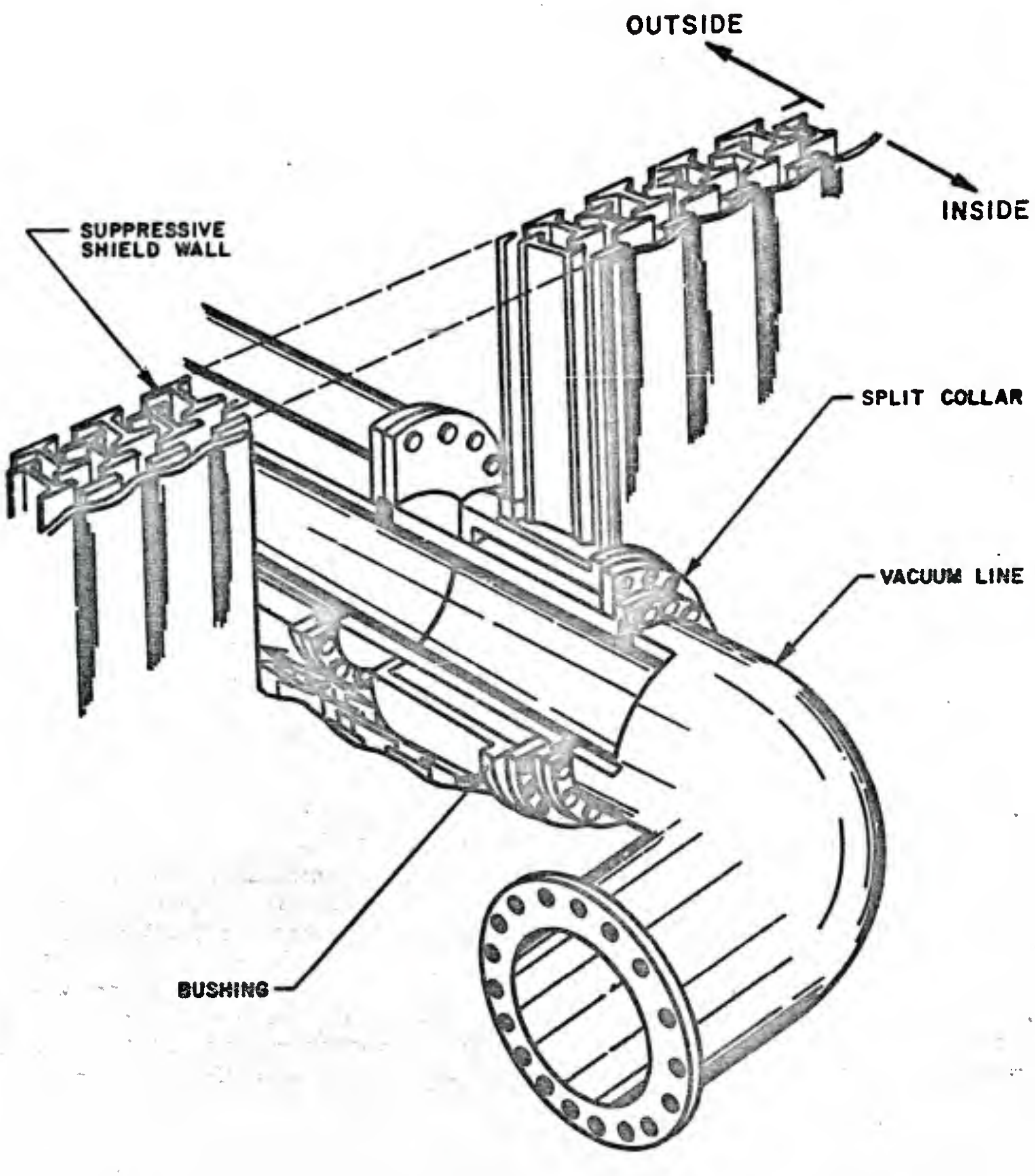


Figure 6-23 Typical vacuum line penetration

6-26.5 *Access Penetrations*

In the munition plant environment, suppressive shields are designed to protect Category I or II hazardous operations as defined in MIL STD 882A.
Remote operation may be required so personnel will not be inside the shield during operations. However, personnel access is required to allow for maintenance, repair, and inspection. Further, these doors must provide large openings to enable most equipment to be installed or removed in large subassemblies.

Access is also required for munition components, explosives, and assembled munitions to pass through the suppressive shield. In the case of conveyor transporting systems, consideration must be given to the proper pass-through of the conveyor. Requirements for this type of access depend on the configuration of the munition product, transporting pallets, and conveyors, as well as production rates and other factors unique to each operation. For these reasons, definition of specific design requirements is not possible.

6-26.5.1 *Personnel Door.* Three different types of doors have been developed for use in suppressive shields: sliding, hinged, and double leaf. The hinged door was designed to swing inward. This feature reduces the usable space inside the shield. A sliding door is preferred for personnel access to munition operations. Figure 6-24 illustrates a typical sliding door. This type door is used with the Group 4, 5, and Milan 81 mm shields. The sliding door consists of an entire shield panel suspended from a monorail system. The panel is inside the shield and is not rigidly attached to the column members. Special consideration was given to the air gap between the door panel and the column to assure that excessive pressure leakage would not occur and that fragments could not pass through the gap.

The cylindrical Group 3 shield contains a two-leaf door, hinged at each side. It swings inward as shown in figure 6-25. The door is curved to match the shield wall contour and is fabricated from S5 x 10 I-beams. Pressure loading restraint is provided by the door bearing on the external support rings of the shield at the top and bottom of the door. An external latch provides restraint during rebound of the door.

6-26.5.2 *Product Door.* Only one type of product door has been developed conceptually for use in suppressive shields. It is the rotary, three lobed configuration shown in figure 6-26. The design procedure for this door is described to illustrate the type of analysis required. It can be used as a guide for analysis of similar alternate design concepts for product doors.

The air blast will most severely load the rotating product door when the munition opening is coincident with the pocket in the rotary door. A non-overriding clutch prevents the door from counter-rotating. The angular impulsive load is:

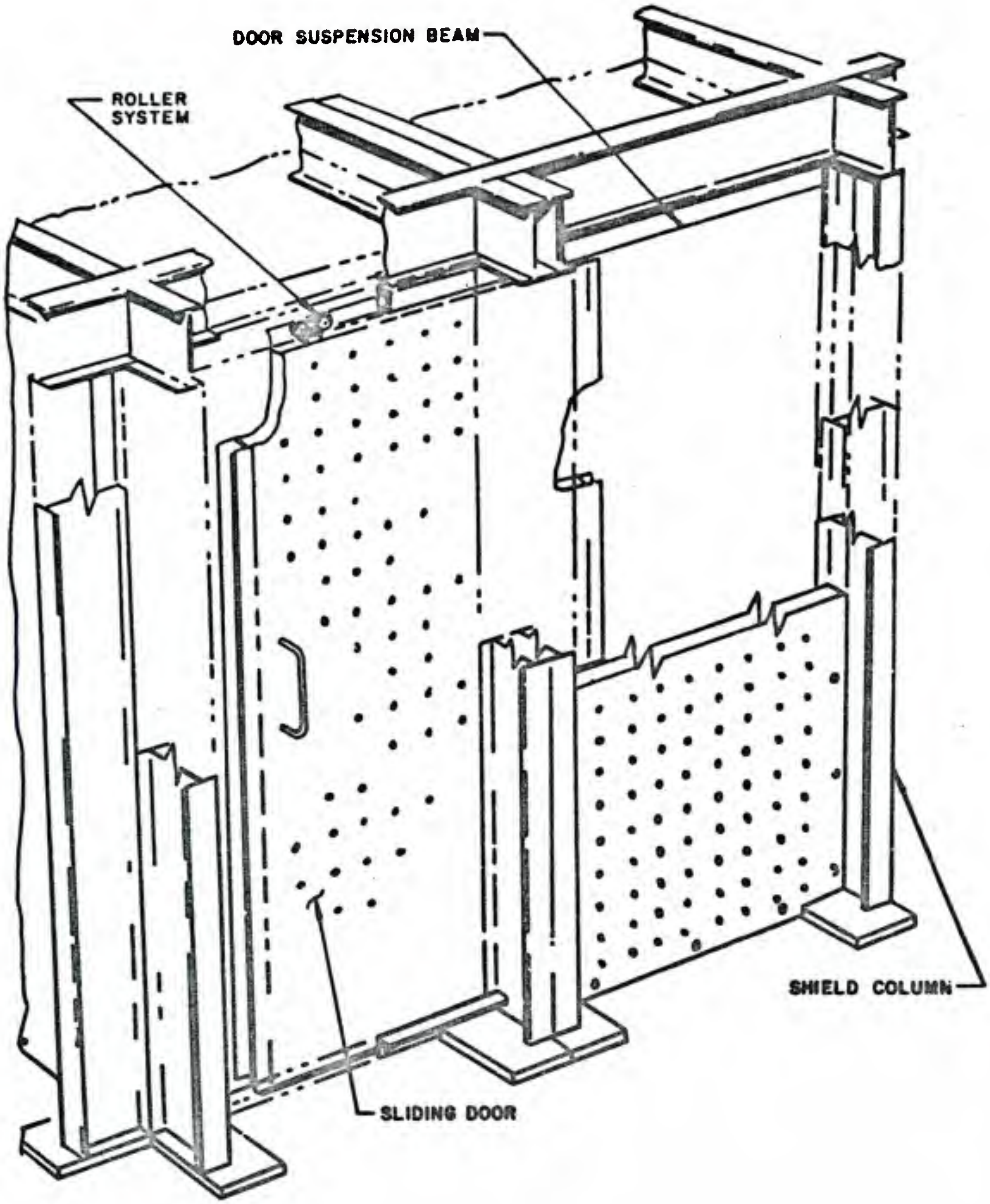


Figure 6-24 Sliding personnel door

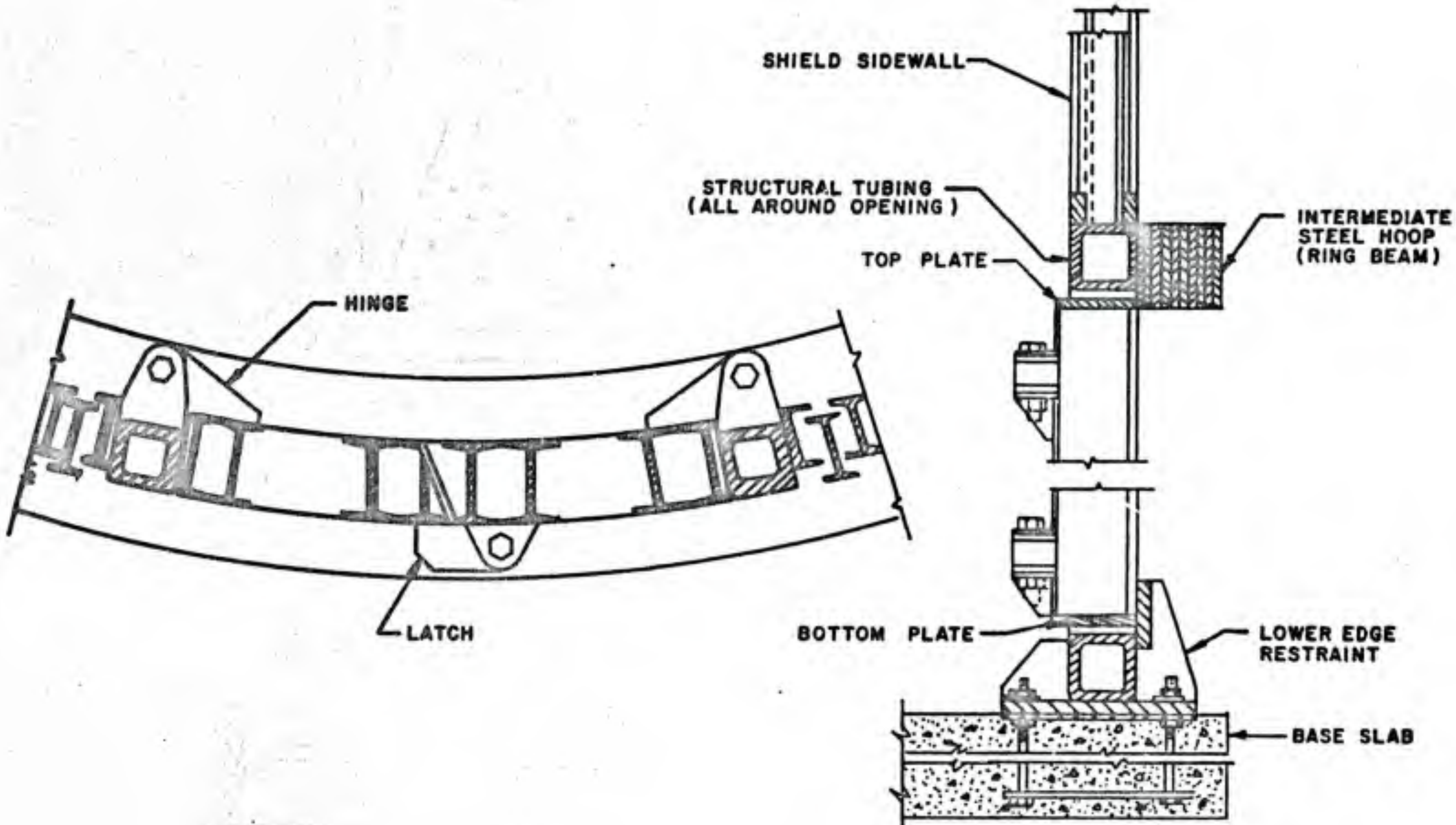


Figure 6-25 Door - group 3 shield

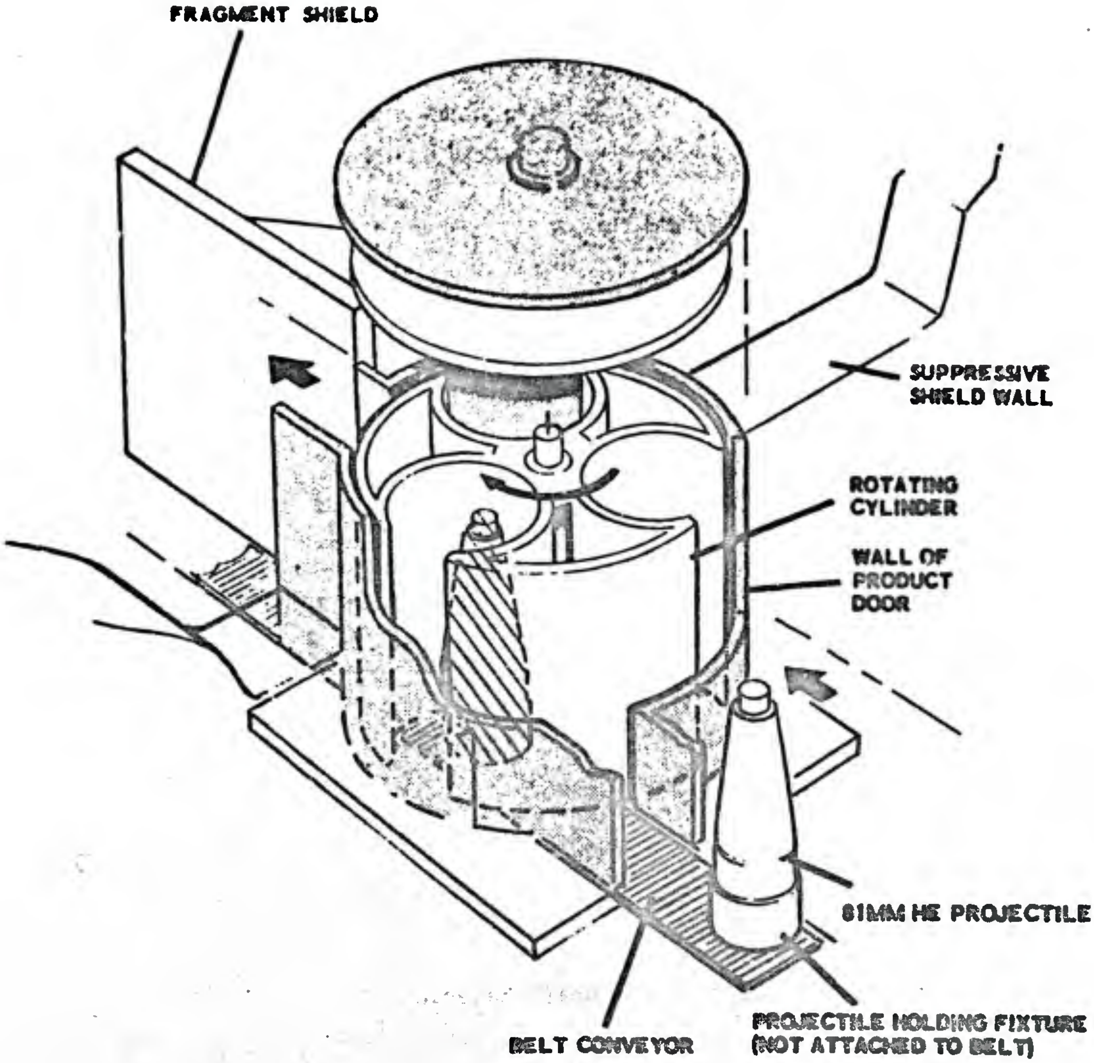


Figure 6-26 Rotating product door

$$T_i = i_r A_d r_d \qquad 6\text{-}38$$

where

T_i = angular impulsive load
i_r = reflected impulse
A_d = door area
r_d = radius from center of impulse load to the center of door rotation

Assuming the product door to be initially at rest, the rotational velocity imparted to the door is given by:

$$\omega = \frac{T_i}{I_m} \qquad 6\text{-}39$$

where

ω = angular velocity
I_m = mass moment of inertia of the door about shaft axis

The kinetic energy imparted to the door is given by:

$$KE = \frac{I_m \omega^2}{2} = \frac{T_i^2}{2I_m} \qquad 6\text{-}40$$

The strain energy absorbed by a circular shaft is given by:

$$U_s = \frac{\pi L_s}{4G} (r_s \tau_s)^2 \qquad 6\text{-}41$$

where

U_s = strain energy

L_s = length of the shaft

G = shear modulus of the shaft material

r_s = radius of the shaft

τ_s = maximum shear stress in the shaft

Equating the kinetic energy of the rotating door to the strain energy in the shaft and solving for the shear stress yields:

$$\tau_s = \frac{T_i}{r_s} \sqrt{\frac{2G}{\pi I_m L_s}} \qquad 6\text{-}42$$

The computed shear stress in the shaft must be less than the dynamic shear stress of the shaft material, i.e.,:

$$\tau_s < 0.55 f_{dy} \qquad 6\text{-}43$$

BLAST RESISTANT WINDOWS

6-27 General

Historical records of explosion effects demonstrate that airborne glass fragments from failed windows are a major cause of injuries from accidental explosions. This risk to life is accentuated in modern facilities, which often have large areas of glass for aesthetic reasons.
Guidelines are presented for both the design, evaluation, and certification of windows to safely survive a prescribed blast environment described by a peak triangular-shaped pressure-time curve. Window designs based on these guidelines can be expected to provide a probability of failure at least equivalent to that provided by current safety standards for safely resisting wind loads.

The guidelines, which apply for peak blast reflected overpressures less than about 20 psi, are presented in the form of load criteria for the design of both the glass panes and framing system for the window. The criteria cover a broad range of design parameters for rectangular-shaped glass panes:

1. Pane aspect ratio - $1.00 \leq a/b \leq 2.0$
2. Pane area - $1.00 \leq ab \leq 25\ ft^2$
3. Pane thickness - $1/8 \leq T_g \leq 1/2$ inch

where

a = long span of pane
b = short span of pane
T_g = thickness of glass

The criteria are primarily for windows which are fully heat-treated, tempered glass. However, criteria are also presented for annealed glass in order to assess the life safety of existing windows that were not originally designed to resist blast overpressures.

6-28 Design Criteria for Glazing

6-28.1 *Glazing Materials*

Fully tempered glass and annealed glass as covered by this design criteria must conform to the requirements of Federal Specifications DD-G-1403B and DD-G-451d., respectively. Tempered glass must also meet the heat-treating requirements of ANSI Z97.1-1975.

Annealed glass is the most common form of glass available today. It is also referred to as plate, float or sheet glass. During manufacture, it is cooled slowly and the process results in very little, if any, residual compressive surface stress. Consequently, annealed glass is of relatively low strength when compared to tempered glass. Furthermore, it has large variations in strength and generally fractures into very sharp fragments. For these reasons, annealed glass is not recommended for use in blast resistant windows.

Heat-treated, tempered glass is the most readily available tempered glass on the market. It is manufactured from annealed glass by heating to a high uniform temperature and then applying controlled rapid cooling. As the internal temperature profile relaxes towards uniformity, internal stresses are created. The outer layers, which cool and contract first, are set in compression, while internal layers are set in tension. As it is rare for flaws, which act as stress magnifiers, to exist in the interior of tempered glass sheets, the internal tensile stress is of relatively minimal consequence. As failure originates from tensile stresses exciting surface flaws in the glass, precompression permits a larger load to be carried before the net tensile strength of the tempered glass pane is exceeded. Tempered glass is typically four to five times stronger than annealed glass.

The fracture characteristics of tempered glass are superior to annealed glass. Due to the high strain energy stored by the prestress, tempered glass will fracture into small cubical-shaped fragments instead of the very sharp fragments associated with fracture of annealed glass.

Semi-tempered glass is often marketed as safety or heat-treated glass. However, it exhibits neither the dicing characteristics upon breakage nor the higher tensile strength associated with fully tempered glass, and, therefore, it is not recommended for blast resistant windows.

Another common glazing material is wire glass, annealed glass with an embedded layer of wire mesh. Wire glass has the fracture characteristics of annealed glass. It also presents metal fragments as an additional hazard. Wire glass is not recommended for blast resistant windows.

The design criteria for blast resistant windows presented here is restricted to heat-treated, fully tempered glass meeting both Federal Specification DD-G-1403B and ANSI Z97.1-1975. Tempered glass meeting only the requirements of DD-G-1403B may possess a surface precompression of only 10,000 psi which can result in a fracture pattern that is similar to annealed and semi-tempered glass, and therefore is not suitable blast resistant windows.

6-28.2 *Design Stresses*

The design stress is the maximum principal tensile stress allowed for the glazing. The design stress has been derived for a prescribed probability of failure using a statistical failure prediction model developed by the American Society of Testing Materials (ASTM). The model accounts for the area of glazing (as it effects the size, number and distribution of surface flaws), stress intensity duration, thickness and aspect ratio of glazing (as it affects the ratio of maximum to minimum principle stresses in the glazing), degree of glass temper (as it affects the precompression stress in the glazing), strength degradation due to exposure, and the maximum probability of

failure required of the glazing. For the full range of design parameters previously sighted, the model predicts a design stress for tempered glass ranging between 16,950 and 20,270 psi based on a probability of failure equal to or less than 0.001 and a load duration of one second. Based on these results, a design stress equal to 17,000 psi has been selected for tempered glass. The model also predicts a design stress for annealed glass ranging between 3,990 and 6,039 psi based on a conventional probability of failure for annealed glass equal to or less than 0.008. Based on these results, a design stress of 4,000 psi has been selected for annealed glass.

These design stresses for blast resistant glazing are higher than those commonly used in the design for one-minute wind loads. However, these higher design stresses are justified on the basis of the relatively short stress intensity duration (less than one second) produced by blast loads.

6-28.3 *Dynamic Response to Blast Load*

An analytical model was used to predict the blast load capacity of annealed and tempered glazings. Characteristic parameters of the model are illustrated in figure 6-27.

The glazing is a rectangular glass plate having dimensions previously mentioned, poisson ratio, $\nu = 0.22$, and modulus of elasticity, $E = 1 \times 10^6$ psi. The pane is assumed to be simply supported along all four edges, with no in-plane and rotational restraints at the edges. The relative bending stiffness of the support elements is assumed to be infinite relative to the pane. The ultimate static stress at failure, f_u, was assumed to be 17,000 psi and 4,000 psi for tempered and annealed glass, respectively.

The blast pressure loading is described by a peak triangular-shaped pressure-time curve as shown in figure 6-27. The blast pressure rises instantaneously to a peak blast pressure, P, and then decays with a blast pressure duration, T. The pressure is uniformly distributed over the surface of the pane and applied normal to the pane. The method for determining the applied blast load is illustrated in Volume II.

The resistance function for the pane accounts for both bending and membrane stresses. The effects of membrane stresses produce a nonlinear stiffness of the resistance-deflection function (fig. 6-27). The ultimate deflection, X_m is defined as the center deflection where the maximum principle tensile stress in the pane first reaches its ultimate stress.

The model utilizes a single degree of freedom system to simulate the dynamic response of the glass (fig. 6-27). Damping of the window pane is included as 5 percent of critical damping. Utilizing the design parameters for the ultimate (or failure) stresses for the glazing as stated above and for specific values of the blast duration, the model calculates the peak blast pressures required to fail the glazing by exceeding the prescribed probability of failure. The model assumes that failure occurs when the maximum deflection exceeds ten times the glazing thickness.

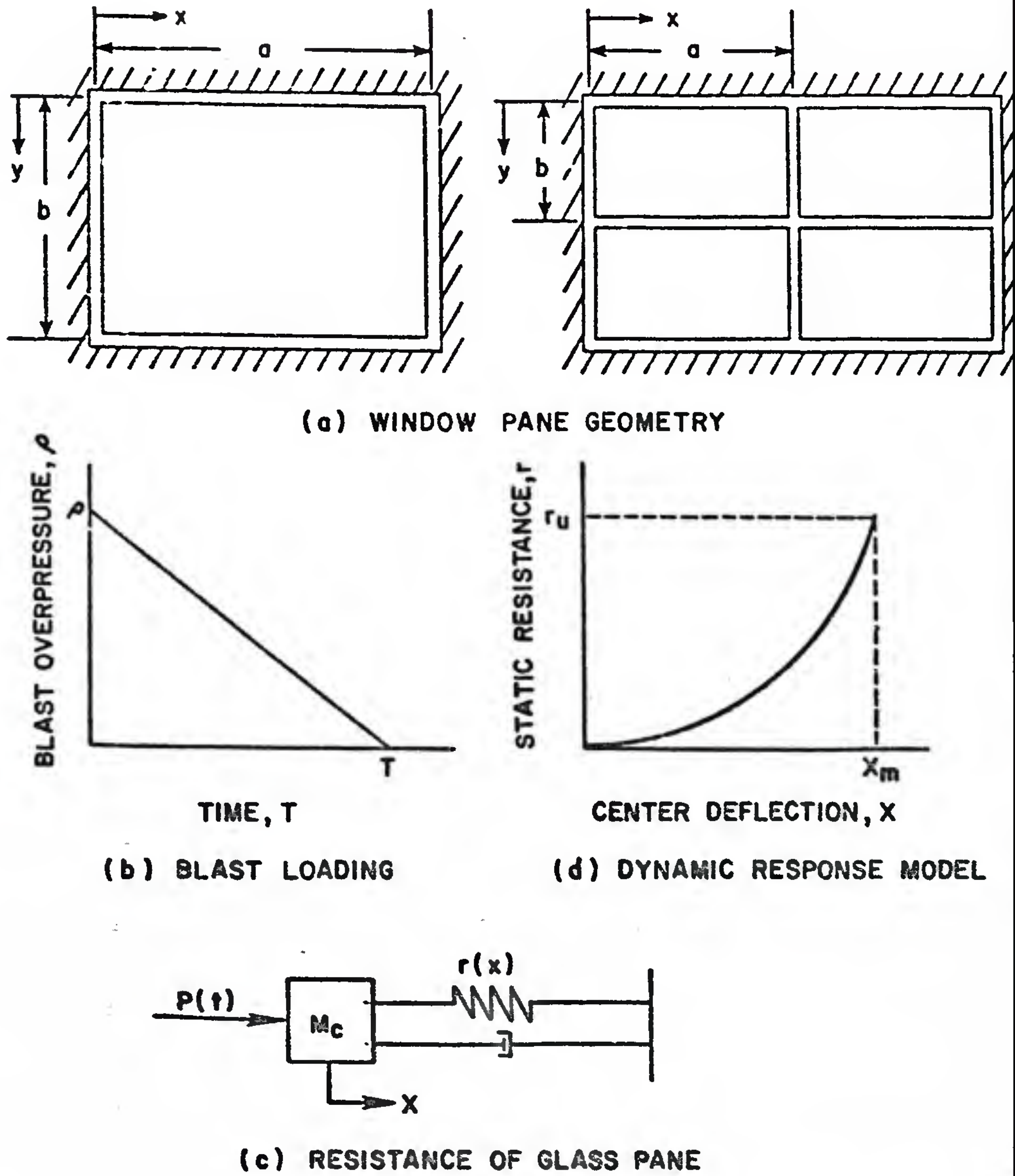


Figure 6-27 Analytical model for glass

6-28.4 *Design Charts*

Based on this design model a series of design charts have been developed and are presented in figures 6-28 to 6-42. (figures are summarized in Table 6-6). These charts relate the peak blast pressure capacity of both tempered and annealed glass for various combinations of window length to height ratios and pane thicknesses. Each chart contains a series of curves for particular pane sizes. For each aspect ratio, the pane size is defined by the short pane dimension b, which is shown at the right of each curve. Figures 6-28 to 6-37 apply to heat-treated tempered glass meeting the requirements of Federal Specification DD-6-1403B and ANSI Z 97.1. Figures 6-38 to 6-42 apply to annealed glass. However, due to the variation in the mechanical properties and fragment hazards involved with annealed glass, these latter charts are not intended for design, but only for safety evaluation of existing glazing.

The charts are based on minimum thicknesses of fabricated glass allowed by Federal Specification DD-G-451d, however, the nominal thickness should be used when using the charts.

The design charts cover a wide range of window geometries. They are presented for panes with aspect ratios of 1.00, 1.25, 1.50, 1.75 and 2.00 and nominal glazing thicknesses of 3/16, 1/4, 3/8 and 1/2 inch for tempered glass and 1/8 and 1/4 inch for annealed glass. The shorter pane dimension b, defining the pane size, ranges from 12 to 60 inches. For windows not conforming to the geometric conditions given, interpolation between charts is necessary. For the required load duration T and shorter pane dimension b, a curve of pressure versus aspect ratio is plotted for each glazing thickness. The required glazing thickness may then be determined for the given pressure and aspect ratio. Due to the limited number of glazing thicknesses available, the minimum glazing thickness required to withstand a given blast loading will, in many cases, be apparent by inspection. In several cases the charts indicate a pane to be slightly stronger than the preceding smaller size. This anomaly is a result of the dynamic effects and the migration of point of maximum principal stresses from the center of the corner region of the pane. In such cases, interpolation should be used between the two curves which bound the desired value.

6-28.5 *Fragment Retention Films*

Most injuries are caused by glass fragments propelled by the blast overpressure when a window is shattered. Commercial products have been developed which offer a relatively inexpensive method to improve the shatter resistance of window glass and decrease the energy and destructive capability of glass fragments. The product is a clear plastic (polyester) film which is bonded to the inside surface of window panes. The film is used primarily for retrofitting previously installed windows. Typical films are about 0.002 to 0.004 inch thick polyester with a self-adhesive face. The film is referred to as shatter resistant or safety film. The film increases life safety by providing a strong, shock absorbing, elastic type backing. The film will hold the glass in position even though the glass is shattered. If a complete pane of film reinforced glass is blown away from its frame by the blast overpressure, it will travel as a single piece while adhering to the film. If

Table 6-6 List of Illustrations for Peak Pressure of Glass

Type	Thickness T_g (in)	LENGTH/HEIGHT RATIO (a/b)				
		1.00	1.25	1.50	1.75	2.00
Tempered	1/2	6-28	6-30	6-32	6-34	6-36
	3/8	6-28	6-30	6-32	6-34	6-36
	1/4	6-29	6-31	6-33	6-35	6-37
	3/16	6-29	6-31	6-33	6-35	6-37
Annealed	1/4	6-38	6-39	6-40	6-41	6-42
	1/8	6-38	6-39	6-40	6-41	6-42

For $1.0 \leq a/b \leq 25$ ft^2,

$12 \leq b \leq 60$ inches, and

$2 \leq T \leq 1000$ ms

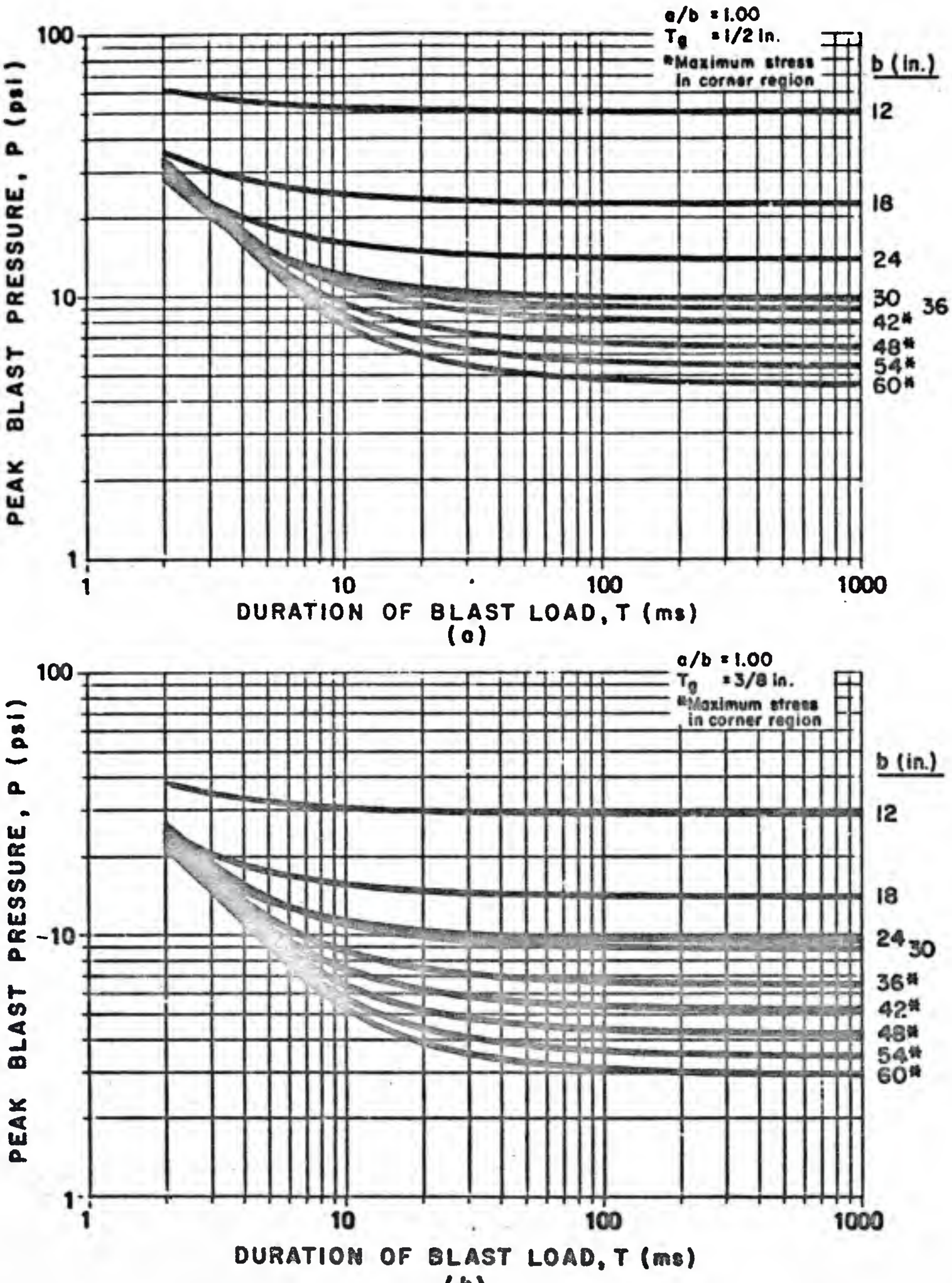


Figure 6-28 Peak blast pressure capacity for tempered glass panes: L/H = 1.00, T_g = 1/2 and 3/8 in.

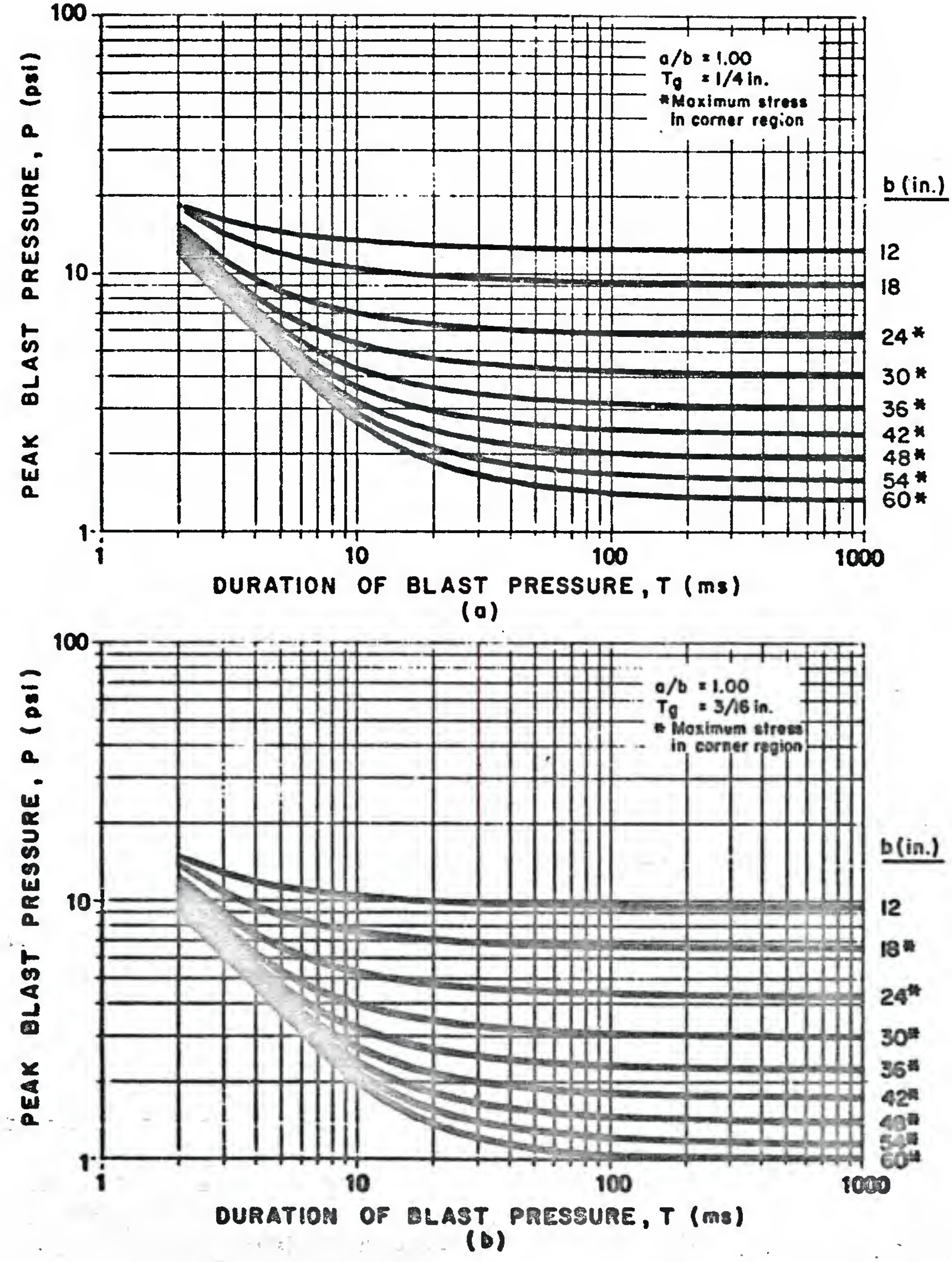


Figure 6-29 Peak blast pressure capacity for tempered glass panes: L/H = 1.00, T_g = 1/4 and 3/16 in.

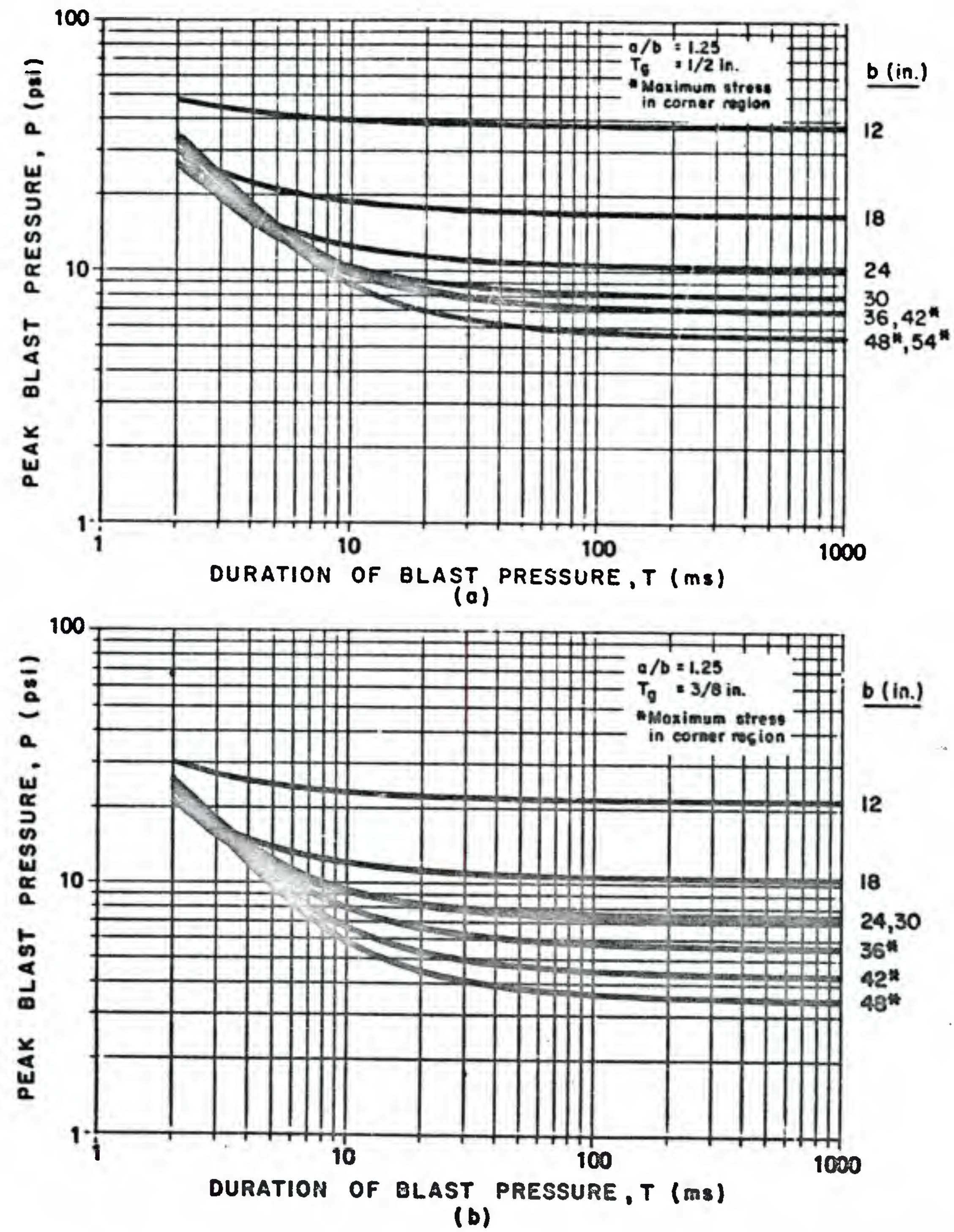


Figure 6-30 Peak blast pressure capacity for tempered glass panes: L/H = 1.25, T_g = 1/2 and 3/8 in.

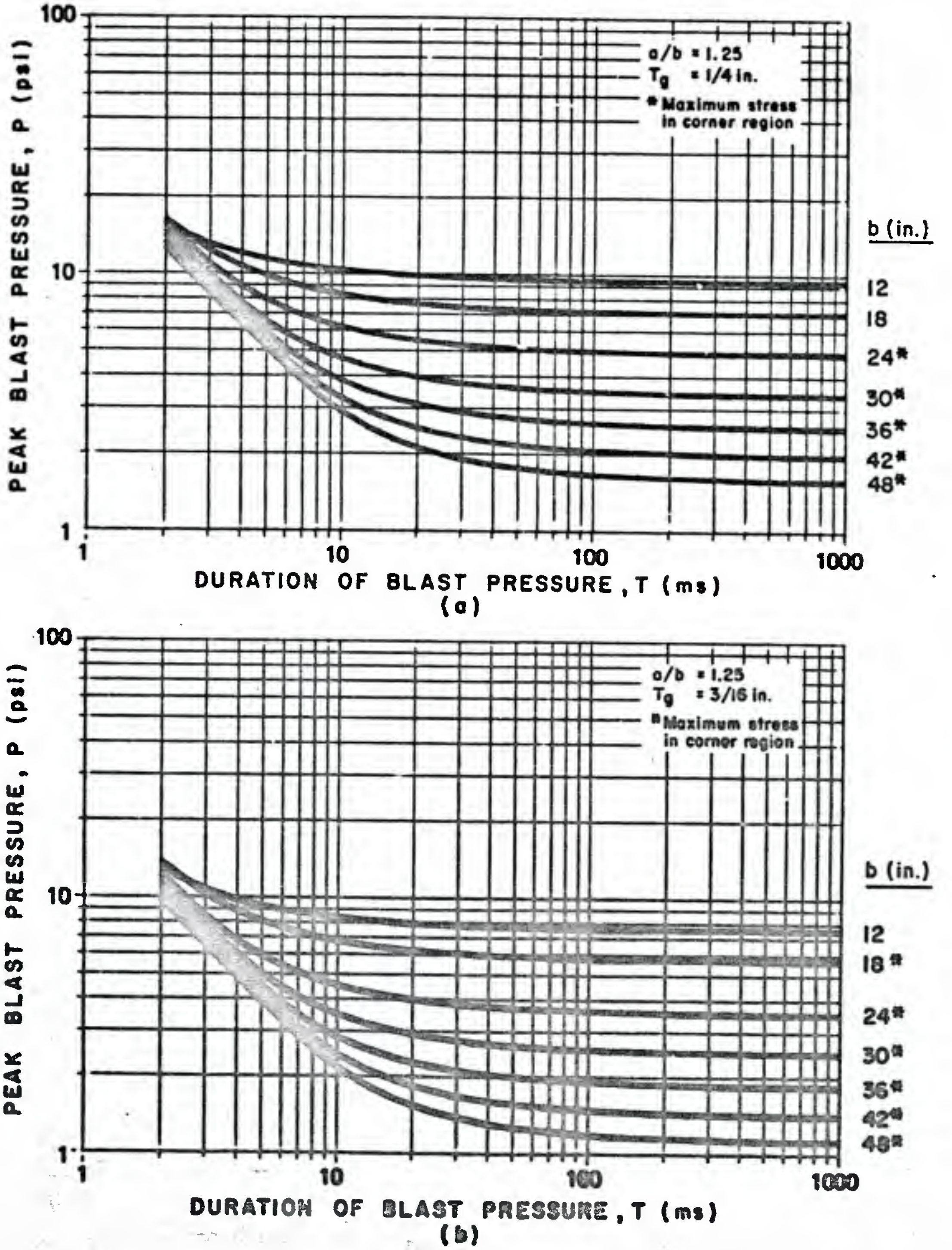


Figure 6-31 Peak blast pressure capacity for tempered glass panes: L/H = 1.25, T_g = 1/4 and 3/16 in.

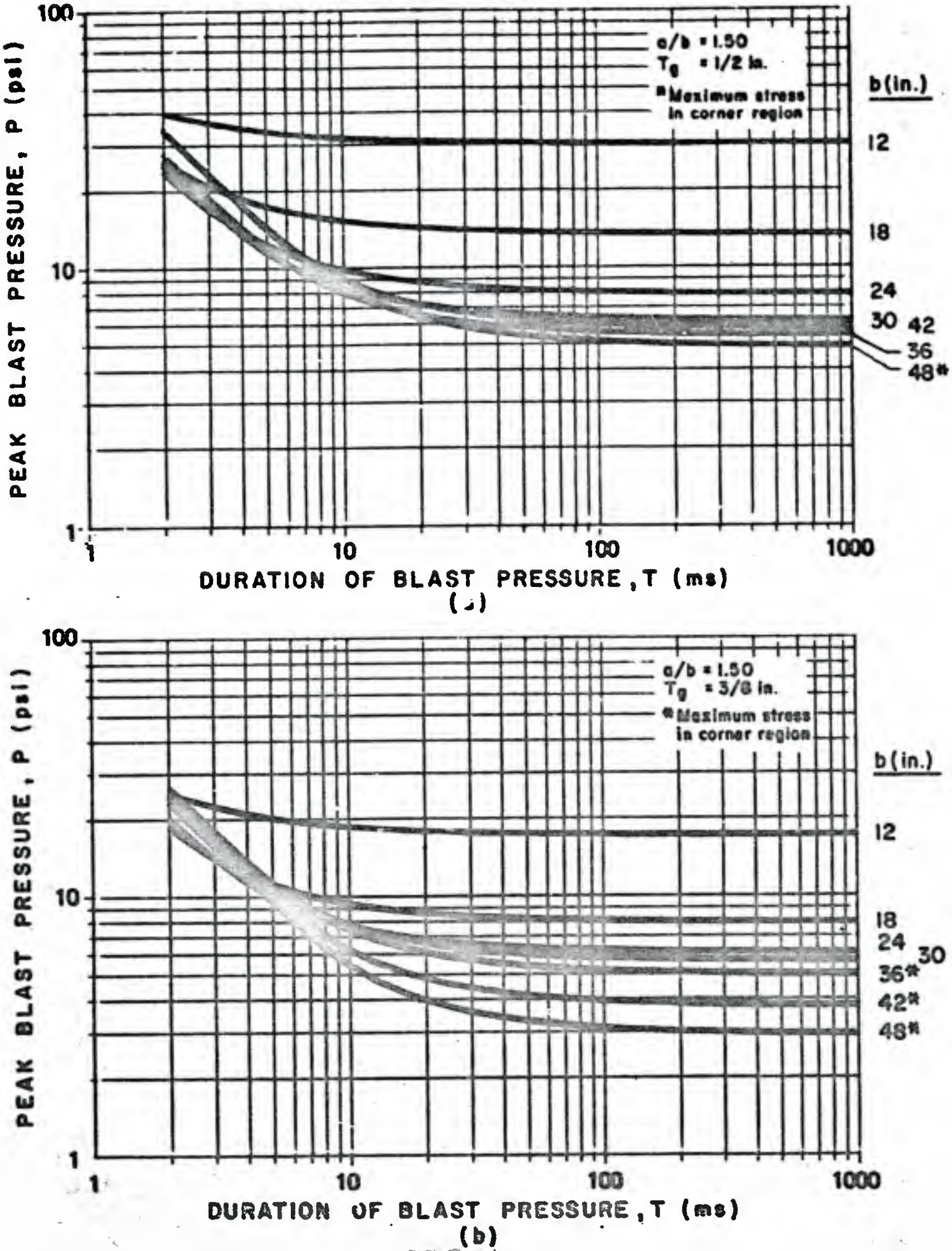


Figure 6-32 Peak blast pressure capacity for tempered glass panes: L/H = 1.50, T_g = 1/2 and 3/8 in.

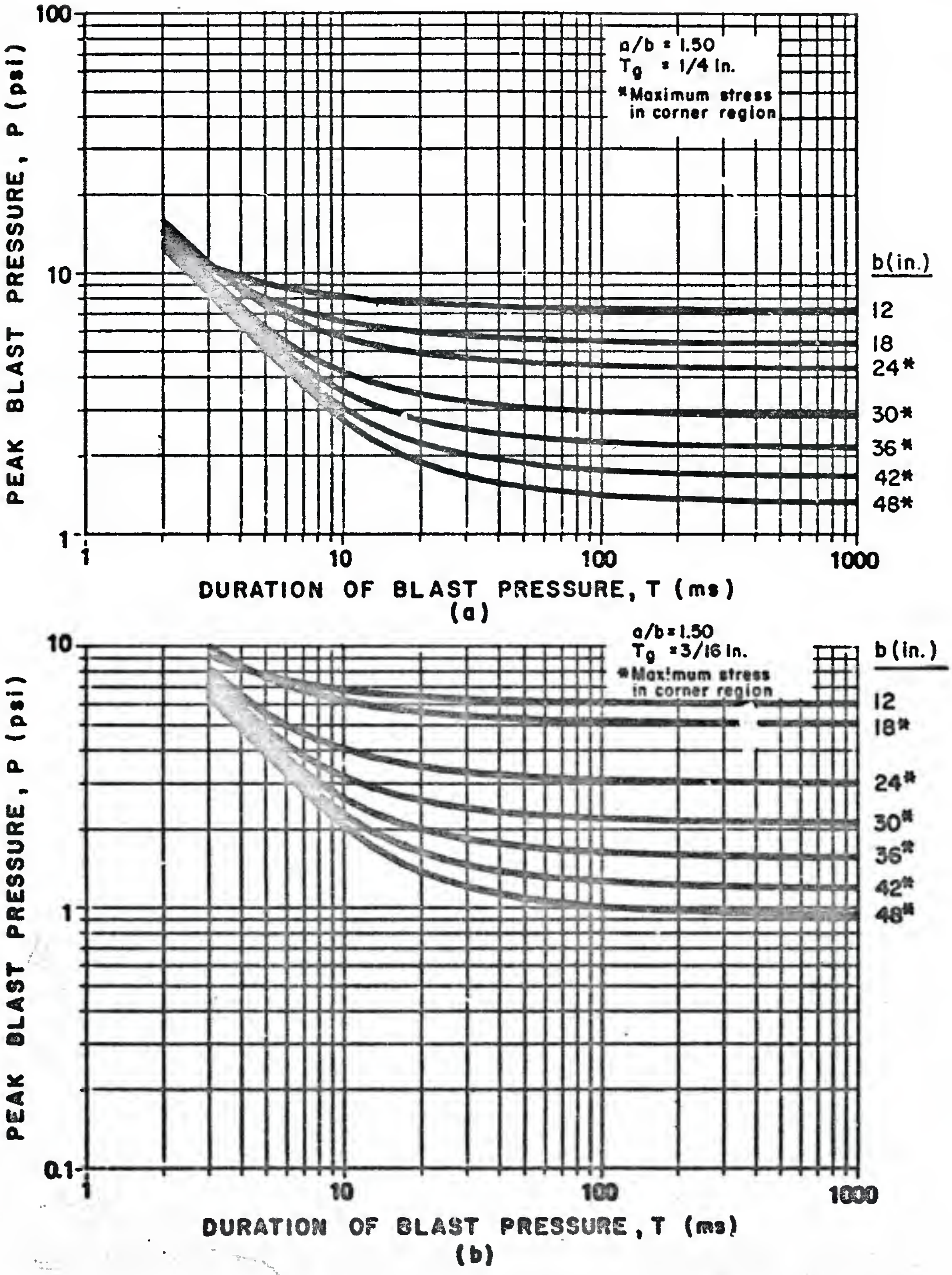


Figure 6-33 Peak blast pressure capacity for tempered glass panes: L/H = 1.50, T_g = 1/4 and 3/16 in.

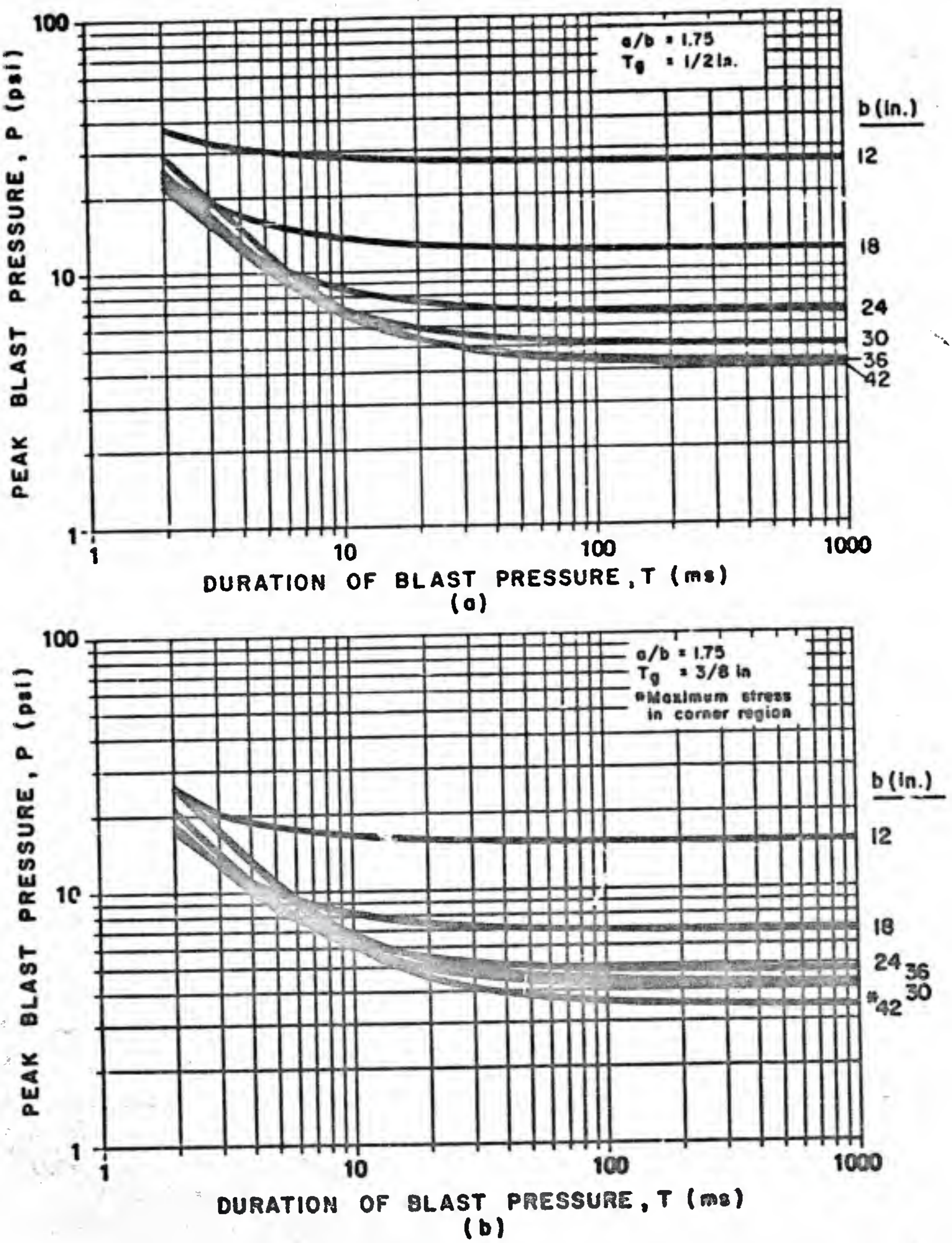


Figure 6-34 Peak blast pressure capacity for tempered glass panes: L/H = 1.75, T_g = 1/2 and 3/8 in.

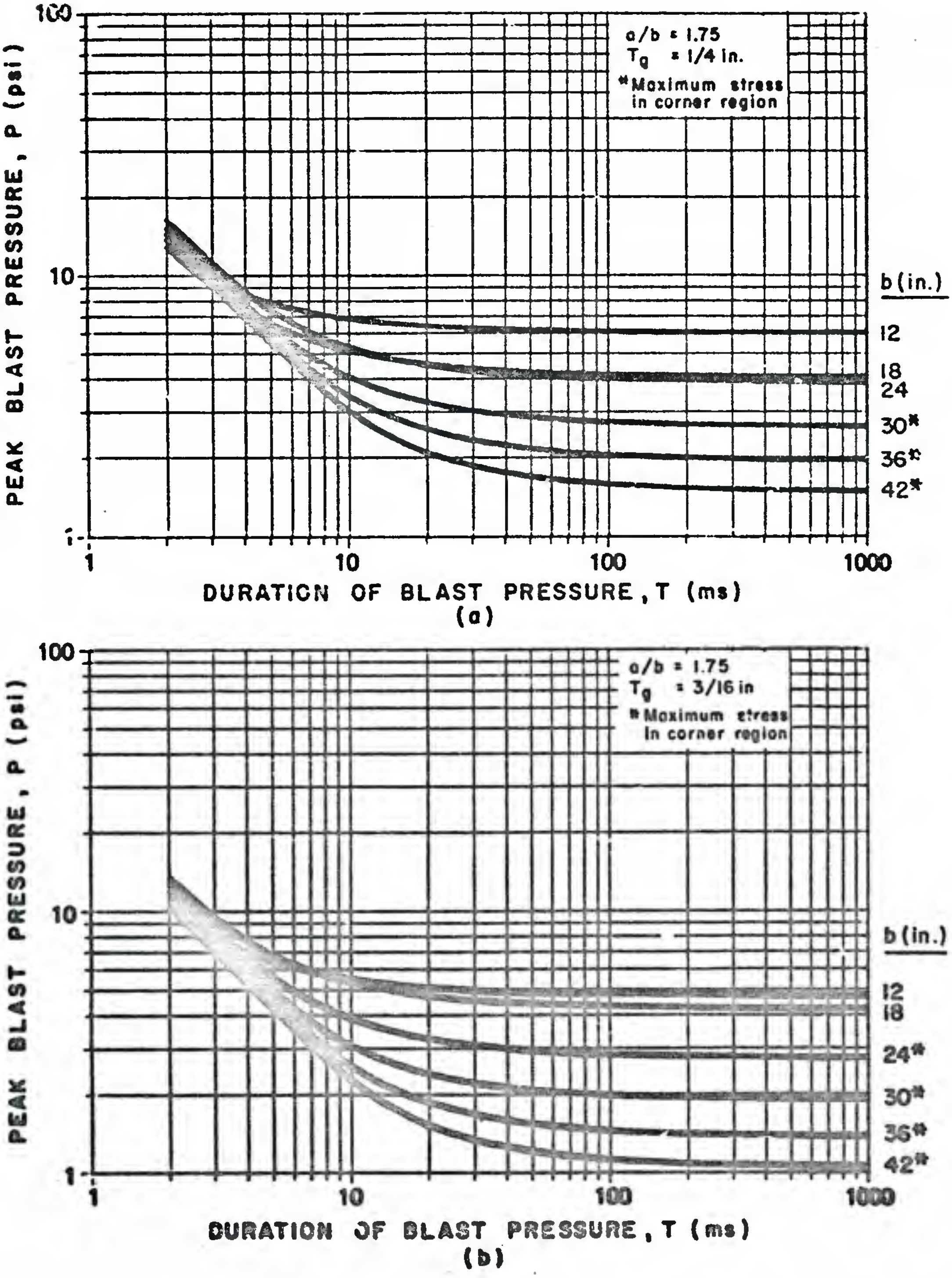


Figure 6-35 Peak blast pressure capacity for tempered glass panes: L/H = 1.75, T_g = 1/4 and 3/16 in.

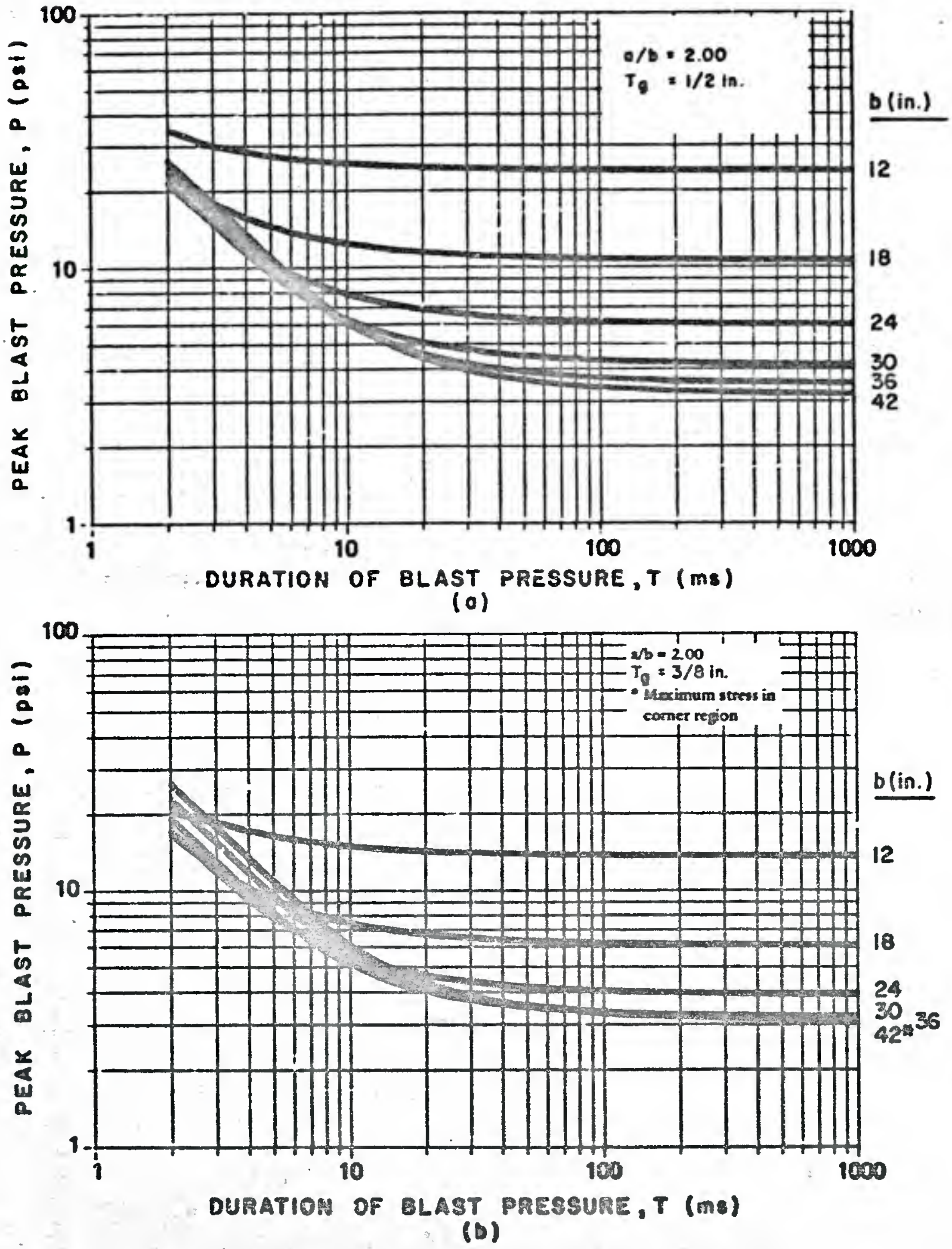


Figure 6-36 Peak blast pressure capacity for tempered glass panes: L/H = 2.00, T_g = 1/2 and 3/8 in.

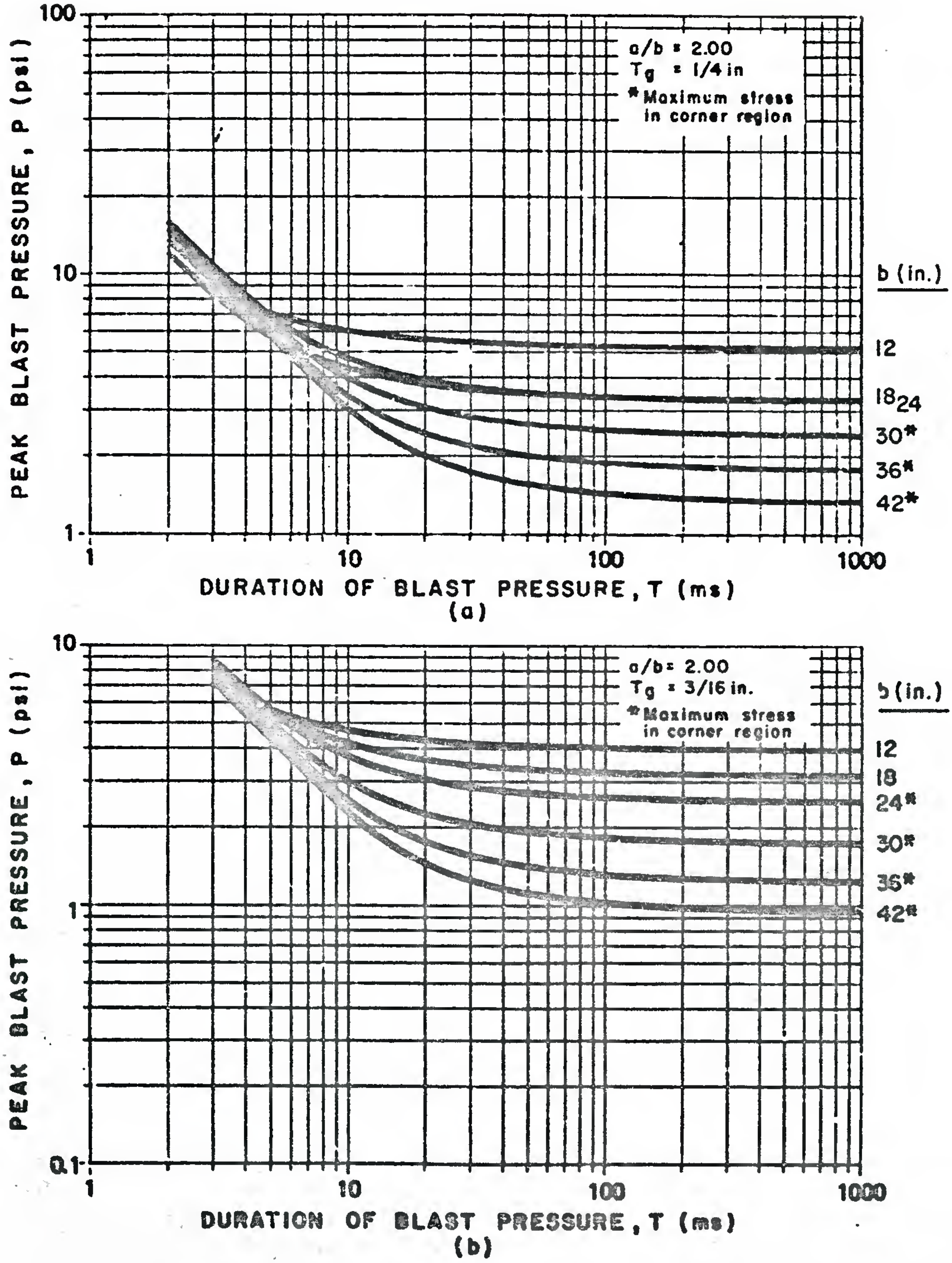


Figure 6-37 Peak blast pressure capacity for tempered glass panes: L/H = 2.00, T_g = 1/4 and 3/16 in.

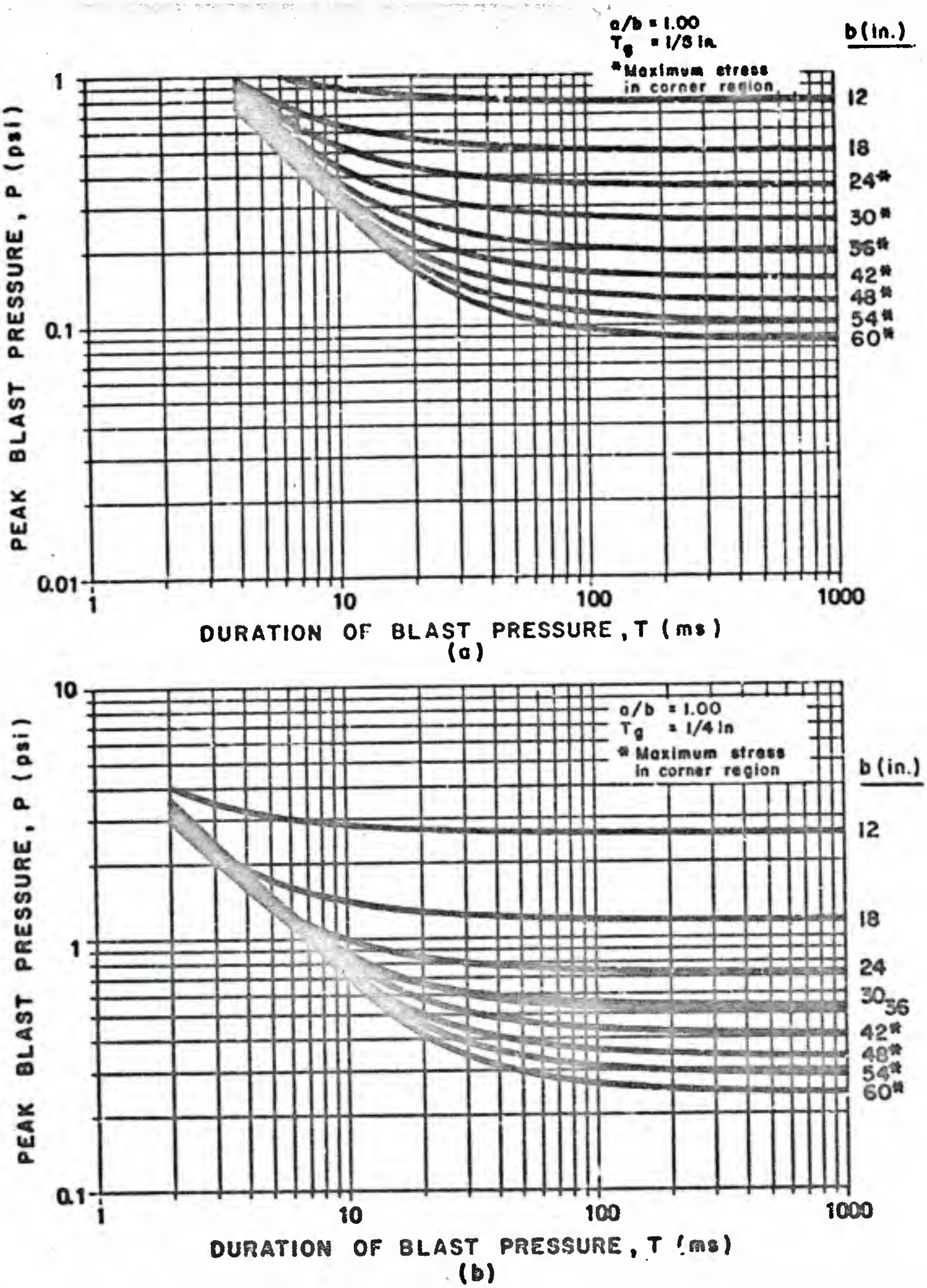


Figure 6-38 Peak blast pressure capacity for annealed glass panes: L/H = 1.00, T_g = 1/8 and 1/4 in.

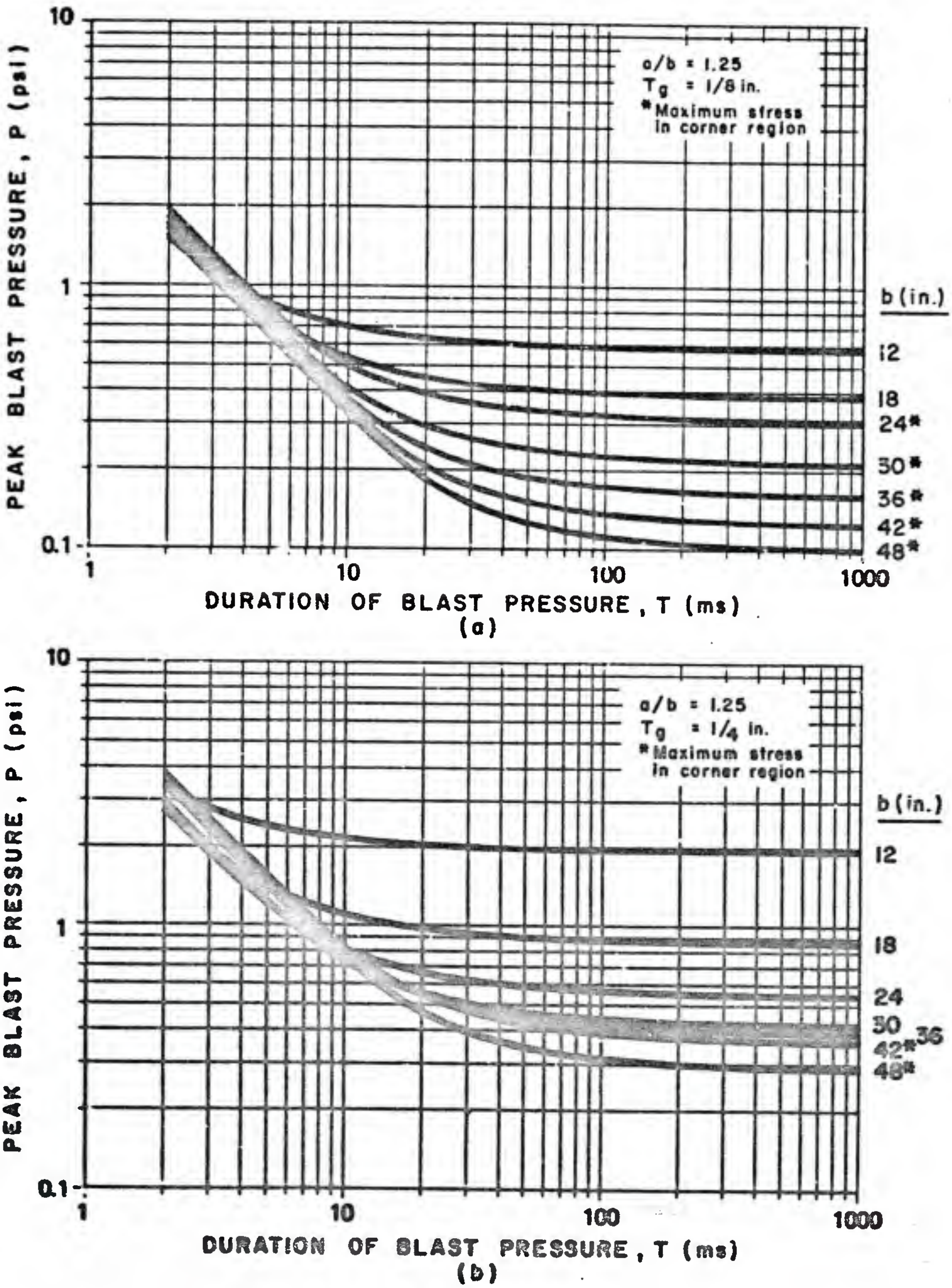
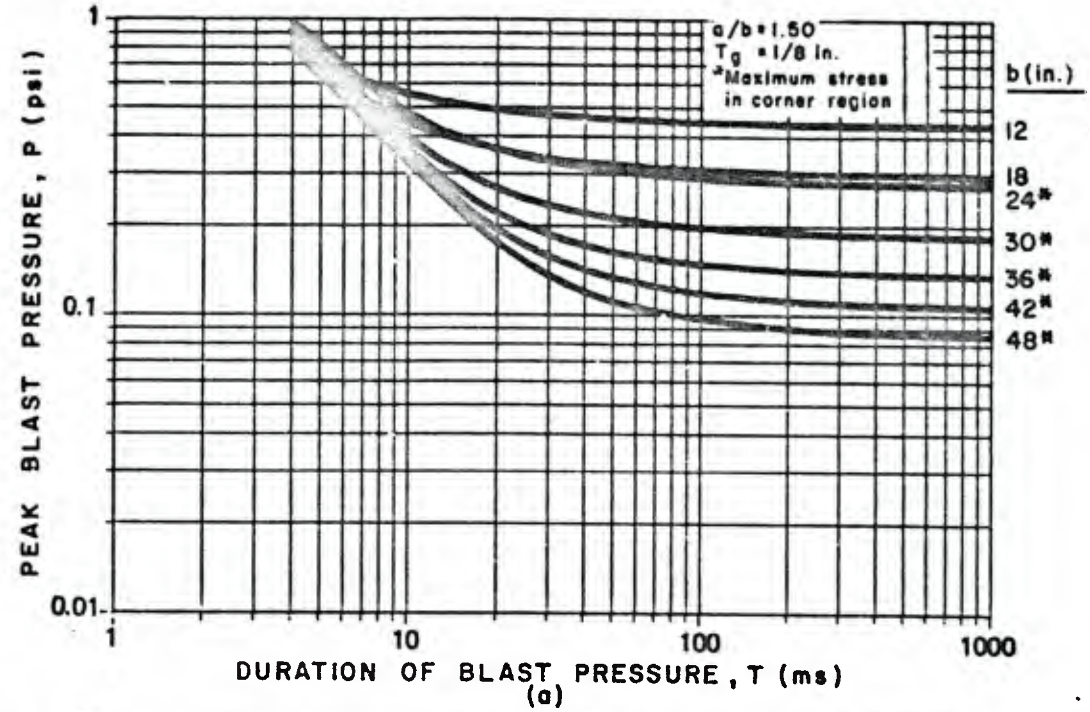


Figure 6-39 Peak blast pressure capacity for annealed glass panes: L/H = 1.25, T_g = 1/8 and 1/4 in.



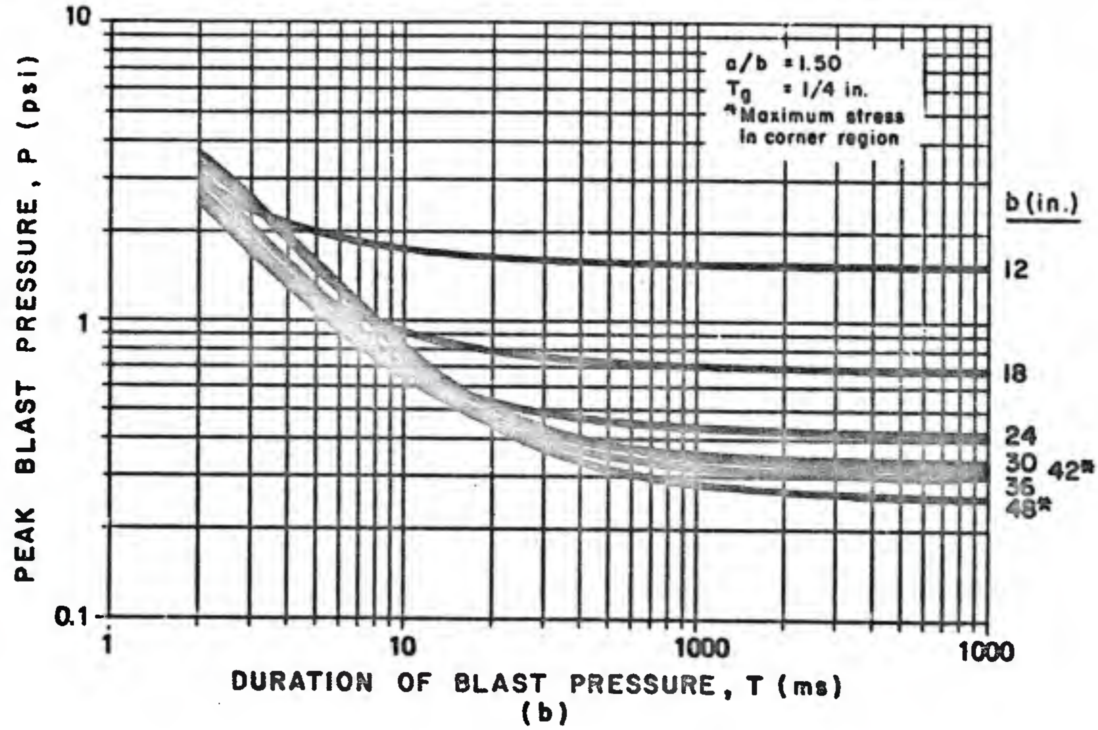
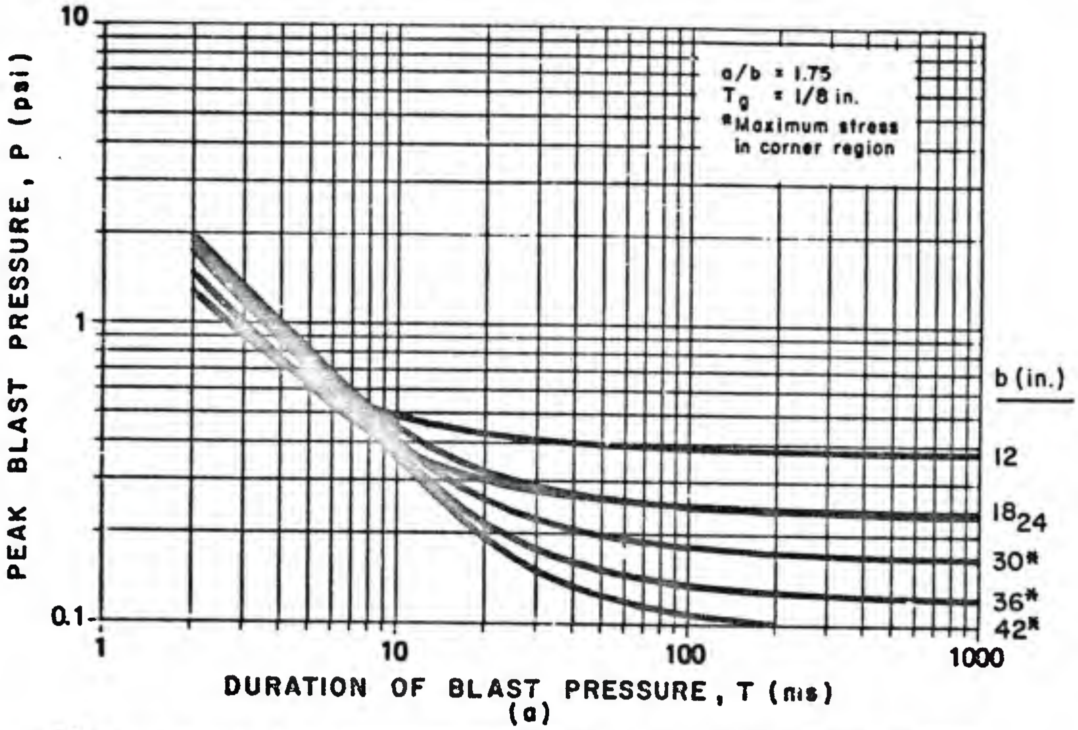
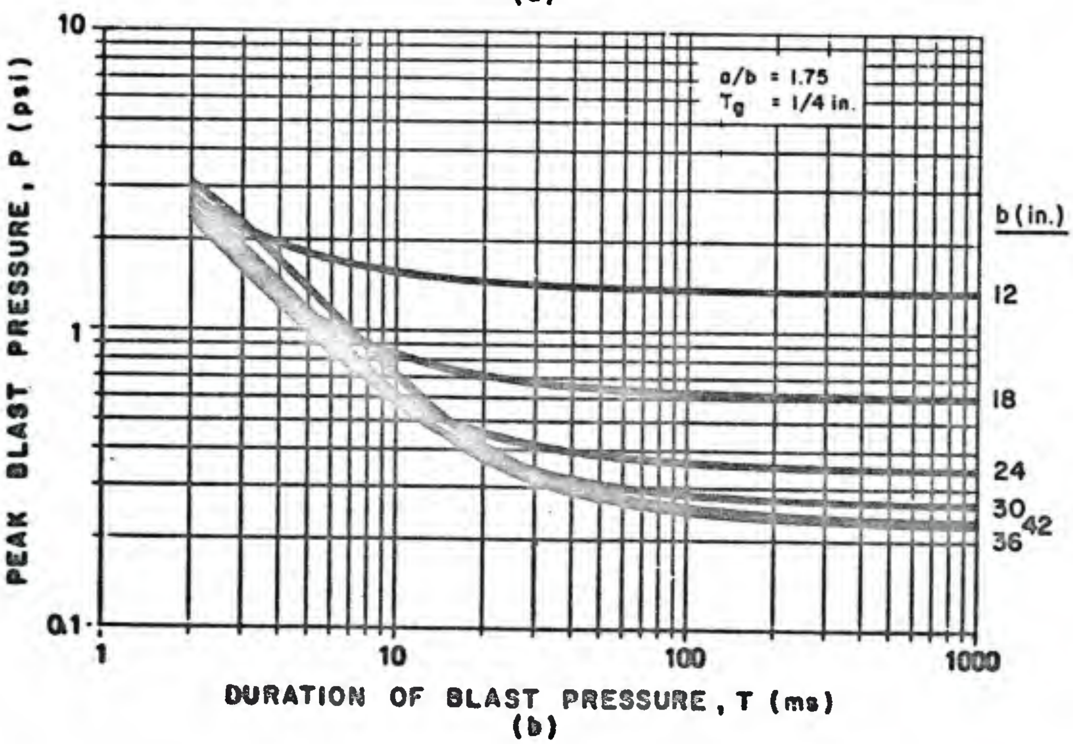


Figure G-40 Peak blast pressure capacity for annealed glass panes: L/H = 1.50, T_g = 1/8 and 1/4 in.

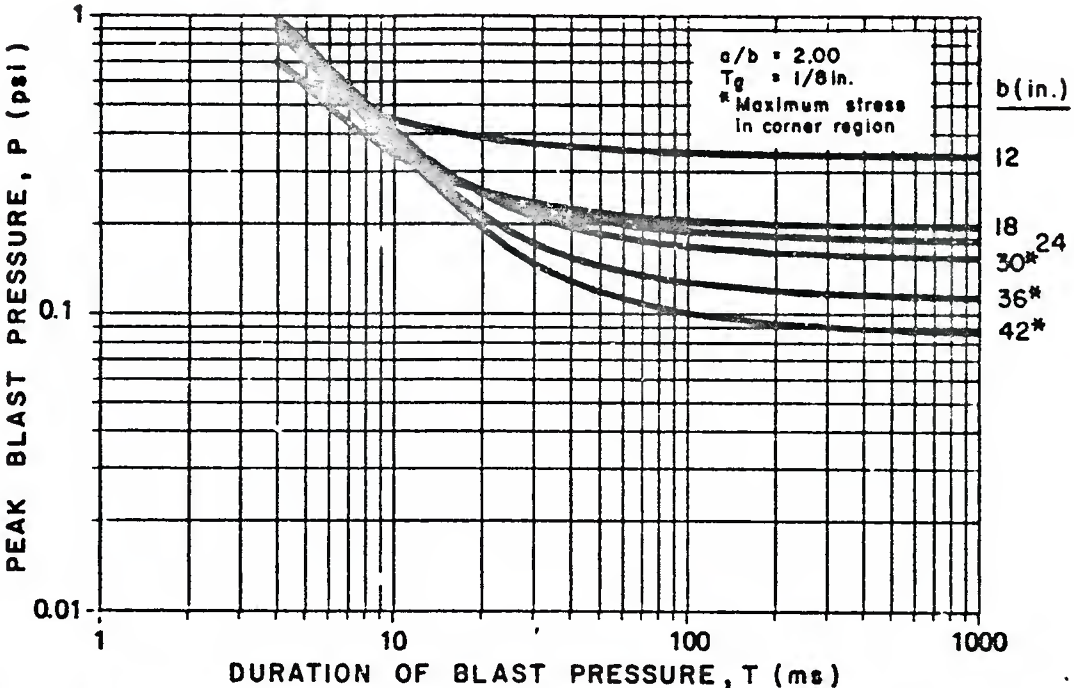


(a)

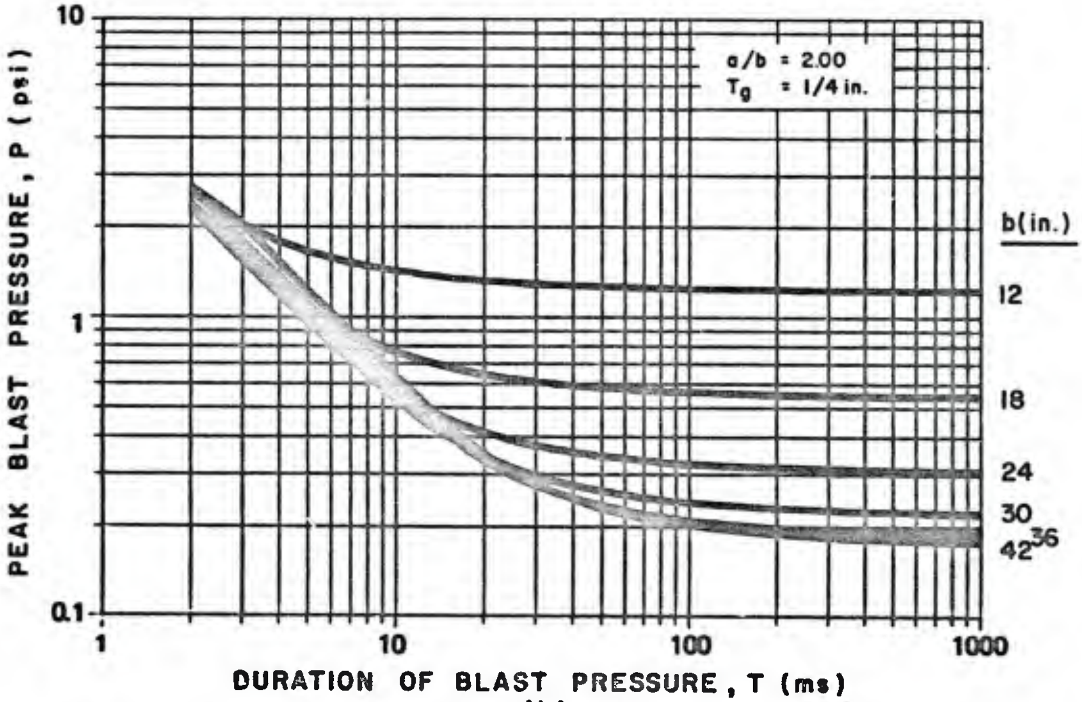


(b)

Figure 6-41 Peak blast pressure capacity for annealed glass panes: L/H = 1.75, T_g = 1/8 and 1/4 in.



(a)



(b)

Figure 6-42 Peak blast pressure capacity for annealed glass panes: L/H = 2.00, T_g = 1/8 and 1/4 in.

a strong structural member or crossbar, which can be decorative, is secured across the opening, the glass will tend to wrap around the crossbar in a manner similar to a wet blanket and will prevented from being propelled across a room. In this configuration it will travel a shorter distance and the individual fragments will be less hazardous because of the shielding effect of the film covering its sharp edges. Additionally, if a projectile strikes the film reinforced glass with sufficient force to pass through it, the glass immediately around the hole will ordinarily adhere to the film. The result is that any fragments broken free by the impact will be few in number and lower in energy content. Results from explosive tests demonstrate that the film is highly effective in reducing the number of air-borne glass fragments. The film can be tinted to improve the heat balance and physical security of the structure. The film also protects the inner tensile surface of the glazing from scratches and humidity, thus reducing strength degradation of the glazing with time.

6-29 Design Criteria for Frames

6-29.1 *Sealants and Gaskets*

The sealant and gasket design should be consistent with industry standards and also account for special requirements for blast resistant windows. The gasket should be continuous around the perimeter of the glass pane and its stiffness should be at least 10,000 psi (pounds/linear inch of frame/inch of gasket deflection). Analysis indicates that the employment of a gasket stiffness below 10,000 psi will increase the failure rate of the window pane. The gasket should provide adequate grip as the glass pane flexes under the applied blast loading. Gaskets shall conform to the requirements of ASTM D-2000.

6-29.2 *Frame Loads*

The window frame shall at a minimum develop the static design strength, r_u, of the glass pane (table 6-7), otherwise, the design is inconsistent with frame assumptions and the peak blast pressure capacity of the window pane predicted from the design charts previously discussed will produce a rate of failure in excess of the prescribed failure rate. This results from the frame deflections which induce higher principal tensile stresses in the pane, thus reducing the strain energy capacity available to resist the blast loading. Presently, there is only qualitative information relating to the interaction of the frames and panes. Therefore, until m. e definitive data becomes available, the following criteria shall be used for design under the action of the applied static strength of the glass listed in table 6-7: (a) individual frame members shall not exceed displacements of 1/264 times their spans or 1/8 inch, whichever is less, (b) stress in any frame member shall not exceed $f_y/1.65$ where f_y is equal to the yield stress of the frame material, (c) the maximum stress in any fastener shall not exceed $f_y/2$ where f_y is the yield strength of the fastener, (d) the stiffness of the frame gasket shall be at least equal to 10,000 psi (pounds/linear inch of frame/inch of gasket deflection).

Table 6-7 Static Ultimate Resistance, r_u, (psi) for Testing Certification of Tempered Glass

b	a/b = 1.00*				a/b = 1.25				a/b = 1.50			
(in.)	T_g=1/2 in.	T_g=3/8 in.	T_g=1/4 in.	T_g=3/16 in.	T_g=1/2 in.	T_g=3/8 in.	T_g=1/4 in.	T_g=3/16 in.	T_g=1/2 in.	T_g=3/8 in.	T_g=1/4 in.	T_g=3/16 in.
12	105.0	60.7	22.6	16.9	80.8	46.7	19.3	13.0	64.5	38.5	13.7	11.3
14	76.7	39.8	17.5	16.2	58.8	33.7	13.5	12.1	46.9	27.5	11.4	10.1
16	58.1	30.5	15.6	13.7	44.7	25.9	11.8	11.2	36.8	20.6	9.95	9.08
18	45.5	24.8	15.3	10.9	34.8	20.9	11.7	9.18	28.5	15.7	9.49	8.09
20	33.2	20.6	12.4	9.38	28.0	17.5	10.5	7.71	22.7	12.5	9.15	5.76
22	27.8	17.4	10.6	8.12	23.4	13.3	8.93	6.62	18.5	11.0	7.88	5.57
24	23.8	15.3	9.35	7.04	20.0	11.9	7.74	5.55	15.9	10.2	6.79	4.73
26	20.7	14.6	8.33	5.57	17.4	11.0	6.80	4.90	13.9	9.34	5.77	4.18
28	18.2	14.5	7.39	4.95	15.5	10.8	6.02	4.34	11.2	9.07	5.03	3.73
30	16.1	14.0	6.59	4.47	12.5	10.7	5.21	3.87	10.4	8.81	4.42	3.32
32	14.7	12.6	5.37	4.05	11.4	10.3	4.71	3.47	9.83	8.54	4.02	2.96
34	14.2	10.9	4.88	3.69	10.7	9.30	4.27	3.13	9.17	8.20	3.67	2.65
36	13.8	9.97	4.47	3.39	10.4	8.42	3.88	2.84	8.79	7.43	3.33	2.34
38	13.7	9.16	4.12	3.12	10.4	7.70	3.54	2.59	8.71	6.78	3.04	2.12
40	13.5	8.55	3.81	2.88	10.3	7.07	3.25	2.31	8.36	6.20	2.77	1.94
42	12.5	7.99	3.54	2.57	10.2	6.53	2.99	2.12	8.23	5.68	2.53	1.79
44	11.0	7.41	3.29	2.45	9.42	6.05	2.76	1.97	8.19	5.11	2.27	1.57
46	10.3	6.90	3.08	2.28	8.72	5.61	2.56	1.83	7.70	4.70	2.10	1.31
48	9.58	6.43	2.88	2.14	8.10	5.06	2.32	1.65	7.16	4.34	1.95	1.11
50	8.97	5.99	2.71	2.01	7.57	4.76	2.16	1.40				
52	8.50	5.58	2.54	1.89	7.09	4.48	2.02	1.20				
54	8.07	4.82	2.37	1.67								
56	7.63	4.55	2.24	1.45								
58	7.25	4.31	2.12	1.26								
60	6.87	4.09	2.01	1.10								

b	a/b = 1.75				a/b = 2.00			
(in.)	T_g=1/2 in.	T_g=3/8 in.	T_g=1/4 in.	T_g=3/16 in.	T_g=1/2 in.	T_g=3/4 in.	T_g=1/4 in.	T_g=3/16 in.
12	57.9	32.7	13.0	9.98	51.1	29.4	11.6	8.57
14	42.2	23.6	10.3	6.85	37.1	21.3	8.94	5.83
16	31.3	17.8	8.58	6.79	28.1	16.2	7.26	5.24
18	24.4	13.9	7.62	6.69	22.0	12.9	5.37	5.15
20	19.5	11.8	7.01	5.81	17.7	10.5	5.07	5.00
22	15.9	10.1	6.73	4.95	14.6	8.86	4.99	4.48
24	13.3	9.03	5.83	4.28	12.3	7.77	4.87	3.85
26	11.8	8.10	5.12	3.76	10.5	6.87	4.64	3.38
28	10.4	7.34	4.50	3.37	9.11	5.34	4.07	2.99
30	9.39	7.08	4.02	3.00	8.20	5.08	3.62	2.66
32	8.66	6.82	3.62	2.63	7.44	4.97	3.26	2.35
34	7.99	6.56	3.31	2.36	6.78	4.86	2.94	2.10
36	7.37	5.30	3.01	2.13	6.21	4.74	2.66	1.88
38	6.93	6.02	2.73	1.94	5.05	4.60	2.42	1.61
40	6.60	5.37	2.46	1.73	4.78	4.53	2.20	1.31
42	6.25	4.94	2.25	1.42	4 60	4.49	2.00	1.08
44	5.93	4.55	2.08	1.18				

* a= longest side of window; b= shortest side of window; T_g= nominal thickness of window.

The design loads for the glazing are based on large deflection theory, but the resulting design loads for the frame are based on small deflection theory for laterally loaded plates. Analysis indicates this approach to be simpler and more conservative than using the frame loading based exclusively on large deflection membrane behavior, characteristic of window panes. According to the assumed plate theory, the static ultimate load, r_u, of the pane produces a unit shear, applied along the long and short side of the pane equal to:

$$V_x = C_x r_u b \sin (\pi x /a) \qquad 6\text{-}44$$

and

$$V_y = C_y r_u b \sin (\pi y /b) \qquad 6\text{-}45$$

where

V_x = unit shear along the long side

V_y = unit shear along the short side

C_x = shear coefficient for the ultimate shear along the long side of frame

C_y = shear coefficient for the ultimate shear along the short side of frame

In equations 6-44 and 6-45, the values of x and y define the coordinates of points along the frame where the shear is being determined. A graphical representation of x and y are illustrated in figures 6-27 and 6-43. The values of C_x and C_y are tabulated in table 6-8 as a function of a/b.

In addition to the shear forces, each frame is subjected to a concentrated load at the four corners which causes uplift of the frame. The magnitude of the uplift force is defined by:

$$R_g = -C_R r_u b^2 \qquad 6\text{-}46$$

where

R_g = uplift force at each corner of frame

C_r = force coefficient

The values of C_R are tabulated in table 6-8 for various values of $^a/b$.

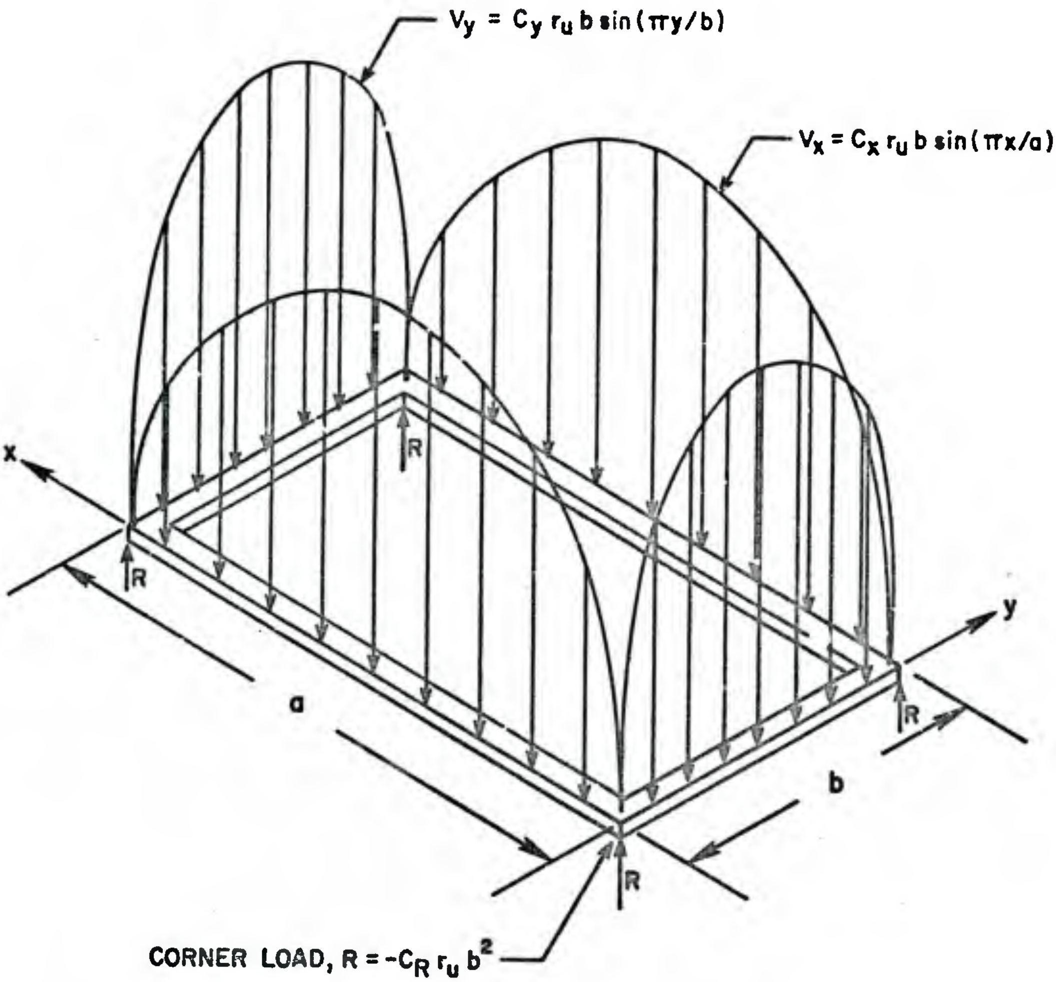


Figure 6-43 Distribution of lateral load transmitted by glass pane to window frame

Table 6-8 Coefficients for Frame Loading

a/b	C_R	C_x	C_y
1.00	0.065	0.495	0.495
1.10	0.070	0.516	0.516
1.20	0.074	0.535	0.533
1.30	0.079	0.554	0.551
1.40	0.083	0.570	0.562
1.50	0.085	0.581	0.574
1.60	0.086	0.590	0.583
1.70	0.088	0.600	0.591
1.80	0.090	0.609	0.600
1.90	0.091	0.616	0.607
2.00	0.092	0.623	0.614

6-29.3 *Rebound*

Vibratory action of windows will result in negative deflections after the maximum positive deflection has been obtained. This negative deflection is associated with negative forces which will require that both glass and frame be anchored to secure the window unit to the structure which otherwise would be propelled outward. However, rebound criteria are not presently available for predicting the equivalent static uniform negative resistance, r_u^-, that a window must safely resist. For long duration loads, the effects of damping will have been accomplished and therefore, rebound effects generally can be neglected for load durations equal to or greater than approximately 400 milliseconds. For short durations, however, damping will not be achieved and, therefore, rebound effects must be considered. The force developed by the rebound may be determined from figure 3-268. However, the use of this chart requires the determination of the period of vibration of glass which may be calculated using equation 3-60. The value of the equivalent stiffness, K_E, is equal to the stiffness of the pane in the elastic range which can be calculated using the deflections obtained from figure 3-36 and the unit resistance of table 6-7. The load-mass factor is determined from table 3-13 while the unit weight of the glass can be taken as equal to 155 pounds per cubic foot. Typical values of the modulus of elasticity, E, and poisson's ratio, ν, may be taken as 10^7 psi and 0.23, respectively.

6-30 Certification

6-30.1 *General*

Certification tests of the entire window assembly are required unless analysis demonstrates that the window design is consistent with assumptions used to develop the design criteria presented in figures 6-28 to 6-42. The certification tests consist of applying static uniform loads on at least two sample window assemblies until failure occurs in either the glass or frame. Although at least two static uniform load failure tests are required, the acceptance criteria presented below encourages a larger number of test samples. The number of samples beyond two may be required by the Owner. All testing shall be performed by an independent and certified testing laboratory with all results tabulated and submitted as part of the calculations.

A probability of failure under testing of less than 0.025 with a confidence level of 90 percent is considered sufficient proof for acceptance and should substantiate a design probability of failure, under the design blast load of 0.001.

6-30.2 *Test Procedure - Window Assembly Test*

The test windows (glass panes plus support frames) shall be identical in type, size, sealant, and construction to those furnished. The test frame assembly shall be secured by boundary conditions that simulate the adjoining structure. Using either a vacuum, a liquid-filled bladder, or a series of wood and styrofoam blocks in combination with a universal testing machine, an

increasing uniform load shall be applied to the entire window assembly (glass and frame) until failure occurs in either the glass or frame. Failure shall be defined as either breaking of glass or loss of frame resistance. The failure resistance of the assembly shall be recorded to three significant figures. The load should be applied at a rate of 0.5 r_u per minute which corresponds to approximately one minute of significant stress duration until failure. Table 6-7 presents the static ultimate resistance of the glass, which correlates with a probability of failure, equal to 0.001 and a static load duration of 1 minute.

6-30.3 *Acceptance Criteria*

The window assembly (frame and glazing) are considered acceptable when the arithmetic mean of all the samples tested, $\bar{r}$, is such that:

$$\bar{r} \geq r_u + s\,\alpha \qquad \text{6-47}$$

where

r_u = static ultimate resistance of the glass pane

s = sample standard deviation

α = acceptance coefficient

For n test samples, $\bar{r}$ is defined as:

$$\bar{r} = \frac{\sum_{i=1}^{n} r_i}{n} \qquad \text{6-48}$$

where r_i is the recorded failure load of the i^{th} test sample. The standard sample deviation, s, is defined as:

$$s = \left[\frac{\sum_{i=1}^{n} (r_i - \bar{r})^2}{(n-1)} \right]^{1/2} \qquad \text{6-49}$$

Convenience in calculation often can be obtained by employing an alternative but equal form of equation 6-49.

$$s = \left[\frac{\sum_{i=1}^{n} r_i^2 - \frac{1}{n}\left(\sum_{i=1}^{n} r_i\right)^2}{(n-1)} \right]^{1/2} \qquad \text{6-50}$$

The minimum value of the sample standard deviation, s, permitted to be employed in equation 6-47 is:

$$s_{min} = 0.145\ r_u \qquad 6\text{-}51$$

This assures a sample standard deviation which is no better than the ideal tempered glass in ideal frames. Care should be exercised so that the sample or (n-1) standard deviation is computed instead of the general population standard deviation. The acceptance coefficient, α, is tabulated in table 6-9 for the number of samples, n, tested.

As an aid to the tester, the following informational equation is presented to aid in determining if additional test samples are justified.

If:

$$\bar{r} \leq r_u + s\ \beta \qquad 6\text{-}52$$

then with 90% confidence, the design will not prove to be adequate with additional testing. The frame should be redesigned and/or thicker glass used. The rejection coefficient, β, is obtained from table 6-9. If the glass assembly is upgraded with thicker glass than required by the design charts (figs. 6-28 through 6-37) to resist a design blast load, it is not required to develop the higher static ultimate resistance of the thicker glass. Instead, a static load equal to twice the design peak blast overpressure, P, shall be resisted by the window assembly. Thus the window assembly with glass, thicker than required, shall be acceptable when:

$$\bar{r} \geq 2\ P + s\ \alpha \qquad 6\text{-}53$$

If equation 6-53 is not satisfied, and:

$$\bar{r} \leq 2\ P + s\ \beta \qquad 6\text{-}54$$

then with 90 percent confidence continued testing will not raise the arithmetic mean of the failure load of the window assembly, $\bar{r}$, to the point of acceptance.

6-30.4 *Rebound Tests*

The window that passes the window assembly test is an acceptable design if the window assembly design is symmetrical about the plane of the glass or if the design blast load duration, T, exceeds 400 msec. Otherwise, the window design must pass a rebound load test to prove that the window assembly can develop the necessary strength to resist failure during the rebound phase of response. The rebound tests shall be conducted using a procedure similar to the window assembly tests, except that the rebound static ultimate resistance shall be substituted for the value of the resistances of equations 6-47, 6-51 and 6-52. Also the uniform resistance is applied to the inside surface of window rather than the exterior surface as is the case with the direct loading. The loading rate for the rebound force shall be the same as the loading rate of the direct loading.

Table 6-9 Statistical Acceptance and Rejection Coefficients

Number of Window Assemblies n	Acceptance Coefficient α	Rejection Coefficient β
2	4.14	.546
3	3.05	.871
4	2.78	1.14
5	2.65	1.27
6	2.56	1.36
7	2.50	1.42
8	2.46	1.48
9	2.42	1.49
10	2.39	1.52
11	2.37	1.54
12	2.35	1.57
13	2.33	1.58
14	2.32	1.60
15	2.31	1.61
16	2.30	1.62
17	2.28	1.64
18	2.27	1.65
19	2.27	1.65
20	2.26	1.66
21	2.25	1.67
22	2.24	1.68
23	2.24	1.68
24	2.23	1.69
25	2.22	1.70
30	2.19	1.72
40	2.17	1.75
50	2.14	1.77

UNDERGROUND STRUCTURES

6-31 Introduction

Underground structures are not usually used for production and handling of explosives since access for both personnel and explosives is more difficult than for an aboveground structure. However, an explosion may result in severe hazards from which an aboveground structure can not provide adequate protection and a buried structure will be required. An example might be a manned control building at a test site which must be located very close to a high-hazard operation involving a relatively large quantity of explosives.

There is limited test data available to predict the pressures acting on an underground structure. What test data that is available was developed for use in the design of protective structures to resist the effects of an attack with conventional weapons. The results of this data and the design procedures developed from it are given in the technical manual, "Fundamentals of Protective Design for Conventional Weapons," TM 5-855-1. The data presented may be expanded to include the design of structures subjected to accidental explosions. The pertinent sections are briefly summarized below.

A typical underground structure used to resist conventional weapons attack is shown in figure 6-44. The burster slab prevents a weapon from penetrating through the soil and detonating adjacent to the structure. A burster slab is not mandatory, but if it is not used the structure will have to be buried much deeper. The burster slab must extend far enough beyond the edge of the building to form at least a 45 degree angle with the bottom edge of the building. It may have to be extended further, though, if it is possible for a bomb to penetrate at a very shallow angle, travel beneath the burster slab and detonate adjacent to the structure (see fig. 6-44). Sand is used as backfill because materials with high volume of air-filled voids and low relative densities are poor transmitters of ground shock. In addition, sand resists penetration better than soil.

6-32 Design Loads for Underground Structures

6-32.1 *General*

The pressure-time relationships for roof panels and exterior walls are determined separately. For the roof panel, an overhead burst produces the most critical loading while for an exterior wall a sideburst is critical. A general description of the procedure for determining the peak pressures and their durations is given below. For a more detailed description, including the required equations, see TM 5-855-1.

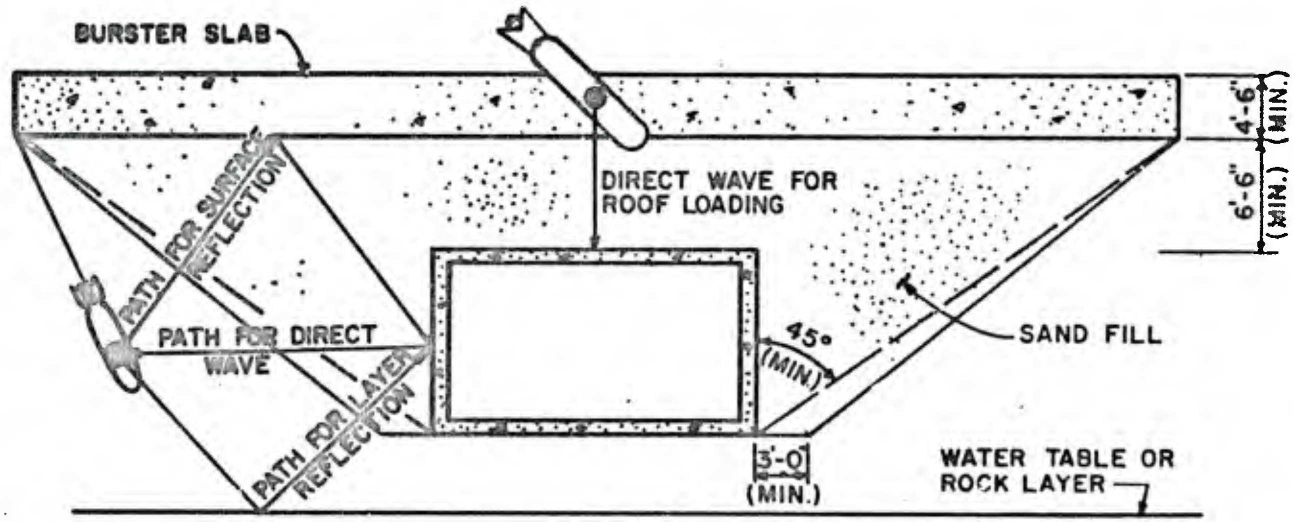


Figure 6-44 Geometry of a buried structure

The magnitude of the ground shock is affected by:

1. The size of the explosive charge and its distance from the structure;

2. The mechanical properties of the soil, rock, and/or concrete between the detonation point and the structure; and

3. The depth of penetration at the time of detonation.

The stresses and ground motions are greatly enhanced as the depth of the explosion increases. To account for this effect a coupling factor is used. The coupling factor is defined as the ratio of the ground shock magnitude from a near surface burst to that of a fully buried burst. A single coupling factor applies to all ground shock parameters and depends on the depth of the explosion and whether detonation occurs in soil, concrete or air.

6-32.2 *Roof Loads*

A typical roof load (shown in fig. 6-45) consists of a free-field pressure P_o and a reflected pressure P_r. The reflected pressure occurs when the free-field pressure impinges on the roof panel and is instantly increased to a higher pressure. The amount of increase is a function of the pressure in the free-field wave and the angle formed between the rigid surface and the plane of the pressure front. However, TM 5-855-1 suggests that an average reflection factor of 1.5 is reasonable.

The pressures on the roof of an underground structure are not uniform across the panel, especially if the depth of the explosion is shallow. However, in order to use a single-degree-of-freedom analysis, a uniform load is required and hence an average uniform pressure must be determined. TM 5-855-1 presents figures that give the ratio peak pressures at the center of a roof panel to the average pressure across the entire panel. This ratio is a function of the support conditions and aspect ratio of the panel and the height of the burst above the roof.

For the most severe roof load the explosive is positioned directly over the center of the panel. The average free-field and average reflected pressures are calculated as described above. The duration of the pressure pulse also varies across the roof panel and a fictitious average duration t_o must be determined. TM 5-855-1 recommends calculating the duration of the peak free-field pressure pulse at a point located one quarter of the way along the short span and at the center of the long span. This duration is then used as the average duration of the entire panel. The peak free-field pressure and impulse are calculated using equations given in TM 5-855-1. The duration is found by assuming a triangular pressure-time relationship. The duration of the average reflected pressure t_r is given in TM 5-855-1 as a function of either the thickness of the structural element or the distance to the nearest free edge of the structure. The smaller of the two numbers should be used in analysis.

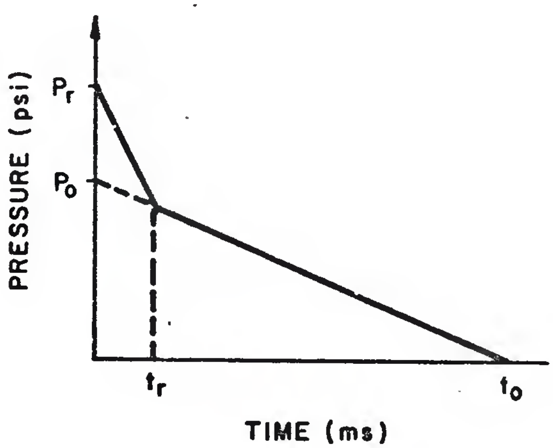


Figure 6-45 Typical roof panel load

6-32.3 *Wall Loads*

The design loads on an exterior wall are determined using the procedures described in Section 6-32.2 for roof loads. However, in addition to the pressure wave traveling directly from the explosion, the wall may be subject to a pressure wave reflected off the ground surface or burster slab and/or a pressure wave reflected off a lower rock layer or water table.

The parameters of each wave (average reflected pressure, average free-field pressure, durations and time of arrival) are determined separately using procedures very similar to those described in Section 6-32.2. The total pressure-time history is equal to the superposition of the three waves as shown in figure 6-46. The superposition results in a very complicated load shape. The response charts of Volume III are not applicable for such a shape, therefore the load must be idealized. The actual load is transformed into a triangular load having the same total impulse (area under the actual load curve equal to the sum of the areas of the direct and reflected waves). The maximum pressure of the idealized load is equal to the maximum pressure of the actual load neglecting the short reflected peaks. The duration is then established as a function of the total impulse and maximum pressure (fig. 6-46). For an exact solution, the actual load curve is used in a single-degree-of-freedom computer program analysis or numerical analysis as given in Section 3-19.2.

6-33 Structural Design

6-33.1 *Wall and Roof Slabs*

The structural design of underground structures is very similar to the design of aboveground structures as described in Volume IV. The effect of the soil is to modify the response of the structural components. The dead load of the soil reduces the resistance available to resist blast. At the same time a portion of the soil acts with the structural elements to increase the natural period of vibration. In the case of a wall, it is assumed that the mass of two (2) feet of soil acts with the mass of the wall. Whereas for a roof, the entire mass of the soil supported by the roof, or a depth of soil equal to one quarter of the roof span (short span for a two-way panel) whichever is smaller, is added to the mass of the roof.

The dynamic response of underground structures must obviously be limited to comparatively small deformations to prevent collapse of the structure due to earth loads. A protective structure subjected to conventional weapons attack should be designed for a ductility ratio of 5.0, as recommended by TM 5-855-1. This ratio may be increased to 10 if special provisions are taken. A maximum deflection corresponding to a support rotation of one (1) degree or a ductility ratio of 10.0 is permitted for underground structures subjected to accidental explosions.

Spalling is the ejection of material from the back face of a slab or beam. It results from high-intensity, close-in explosions. Fragment shields or backing plates, as shown in figure 6-47, are of some value in protecting personnel and

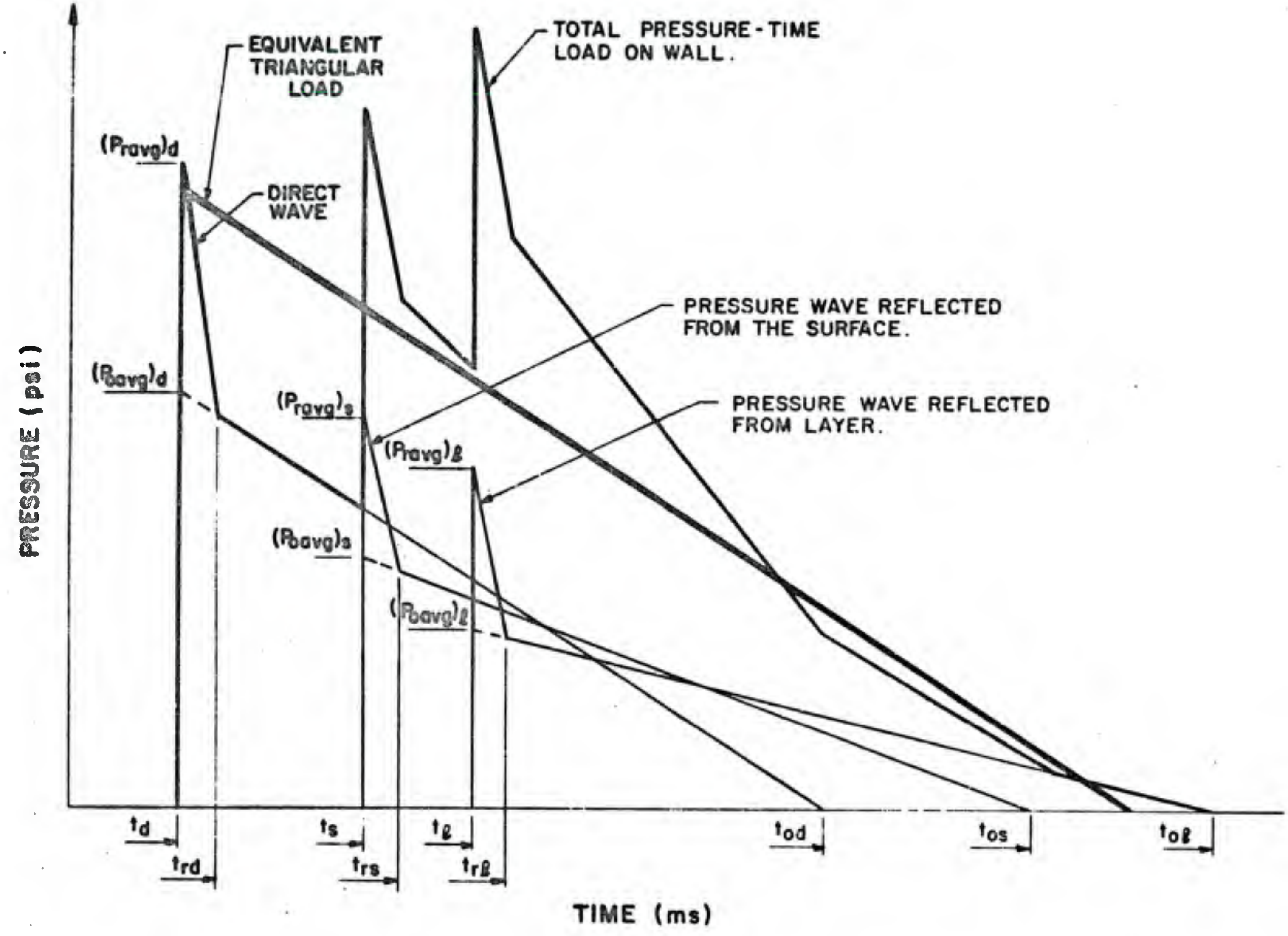
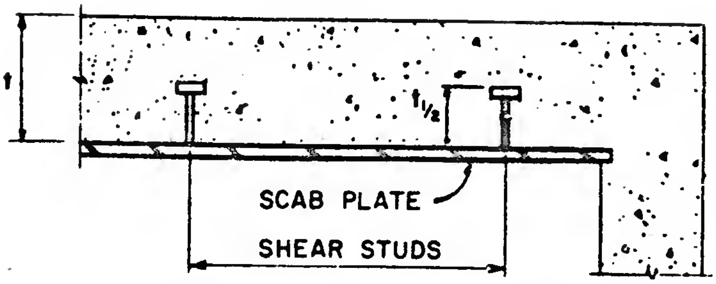


Figure 6-46 Contribution of three pressure waves on a wall



RETENTION OF SCAB MATERIAL

Figure 6-47 Spall plate

equipment. These steel plates must be securely anchored to the inside face of the concrete member. Tests have shown that the shock of a deep penetrating detonation to be enough to cause inadequate welds to fail over a large area, adding the whole steel plate to the concrete spall. A strongly attached plate adds about 10 percent to the perforation resistance of a concrete slab. For a further discussion of backing plates, see Volume IV.

6-33.2 *Burster Slab*

For protective structures, a burster slab prevents a weapon from penetrating through the soil and detonating adjacent to the structure. Its thickness and length may have to be determined by a trial and error procedure in order to limit pressures on the structure to a given value. However, the minimum dimensions are shown in figure 6-44. The minimum reinforcement is 0.1 percent in each face, in each direction or a total of 0.4 percent.

In the design of structures subject to accidental explosions, the ground floor slab of the donor building serves a purpose similar to that of a burster slab. The floor slab helps to prevent fragment penetration and to attenuate the load.

6-34 Structure Motions

6-34.1 *Shock Spectra*

TM 5-855-1 gives equations for acceleration, velocity and displacements for underground structures. These simplified methods take into account the attenuation of the pressure wave as it transverses the structure. For a sideburst, the vertical acceleration, velocity and displacements are 20 percent of the horizontal values. The horizontal motions are uniform over the entire floor while vertical motions at the leading edge are twice those at midspan.

Once the peak in-structure acceleration, velocity and displacements have been determined an in-structure shock spectra can be developed using the principles of Volume II of this manual.

6-34.2 *Shock Isolation Systems*

Volume I presents the upper limits of the shock environment that personnel and equipment can tolerate. If the shock environment exceeds human tolerances and/or equipment "fragility levels" then shock isolation systems are required to protect personnel and sensitive equipment. Using the shock spectra developed as described above, shock isolation systems are designed as outlined in Sections 6-43 through 6-49.

EARTH-COVERED ARCH-TYPE MAGAZINES

6-35 General

Certain types of earth-covered concrete-arch and steel-arch magazines have been approved and standardized for use by the Department of Defense Explosives Safety Board. These magazines provide definite advantages over other types of magazines. Among these advantages are:

1. Less real estate per magazine is required because of the decreased intermagazine separations permitted when approved magazines are used.

2. An almost infinite number of storage situations exists because magazines can be designed to any length.

3. Because of the reduced separation distances, less roads, fences, utilities, etc., are required.

Unlike the other structures discussed elsewhere in this manual, an earth-covered magazine is not designed to resist the damaging effects of its exploding contents. It is accepted that the magazine will be demolished if an internal explosion occurs. During such an incident, the inside of a large-span arch might experience initial blast pressures considerably in excess of 10,000 psi. Less than 100 psi could lift the arch completely out of the ground; therefore, the major portion of the protection is provided by the receiver magazines rather than the donor magazine.

Earth-covered magazines are utilized primarily to prevent propagation of explosion. These structures may also be used for operating buildings and can provide personnel protection. In such cases, separation distances greater than those required to prevent propagation of explosions will be necessary. In addition, a special evaluation of the structure is requried. This evaluation must include the leakage of blast pressures into the protected area, the strength and attachment of easily damaged or lightly supported accessories which may become hazardous debris, the transmission of shock to personnel through the walls or floors, and overall movement of the magazines.

6-36 Description of Earth-Covered Arch-Type Magazines

A typical earth-covered arch-type magazine used for storing explosives has the following features:

1. A semicircular or oval arch constructed of reinforced concrete or corrugated steel used to form roof and sides.

2. A reinforced concrete floor slab, sloped for drainage.

3. A reinforced concrete rear wall.

4. A reinforced concrete headwall that extends at least 2-1/2 feet above the crown of the arch.

5. Reinforced concrete wingwalls on either side of the headwall. The wingwalls may slope to the ground or may adjoin wingwalls from adjacent magazines. The wingwalls may be either monolithic or separated by expansion joints from the headwall.

6. Heavy steel doors in the headwall (either manually operated and/or motorized).

7. An optional gravity ventilation system.

8. Earth cover over the top, sides and rear of the structure. This cover must be at least 2 feet thick at the crown of the arch. The earth above the structure (within the spring line of each arch and between the head and rear walls) is sloped for drainage while beyond the outline of the structure the earth is sloped 2 horizontal to 1 vertical.

9. Its own built-in lightning protection and grounding systems.

A typical earth-covered steel arch magazine is illustrated in figure 6-48.

6-37 Separation Distances of Standard Magazines

Numerous full scale tests of standard magazines have been performed over a period of several years. As a result magazine separation formulae have been established, which will prevent magazine-to-magazine propagation of explosions. All possible right angle arrangements have been considered, i.e., side-to-side, rear-to-rear, front-to-rear, etc. The standard magazines, which are at least equivalent in strength to those tested, are listed in the DoD Standard, "DoD Ammunition and Explosives Safety Standards, 6055.9-STD." The required separation distances, as a function of the quantity of explosives stored in the structure are also given in the DoD Standard. A possible magazine arrangement is shown in figure 6-49.

6-38 Design

Protection of magazines adjacent to a donor magazine is accomplished by combining the following factors:

1. The intensity of the pressure front moving from the donor magazine to receiver magazines diminishes rapidly as the distance traveled increases.

2. The earth cover over and around the donor magazine provides some confinement and tends to directionalize the explosive force both upward and outward from the door end of the magazine.

3. The earth around and over receiver magazines resists fragment penetrations and provides mass to the arch to resist the blast pressures.

4. The arch of receiver magazines is capable of resisting blast loads considerably in excess of the dead loads normally imposed on it.

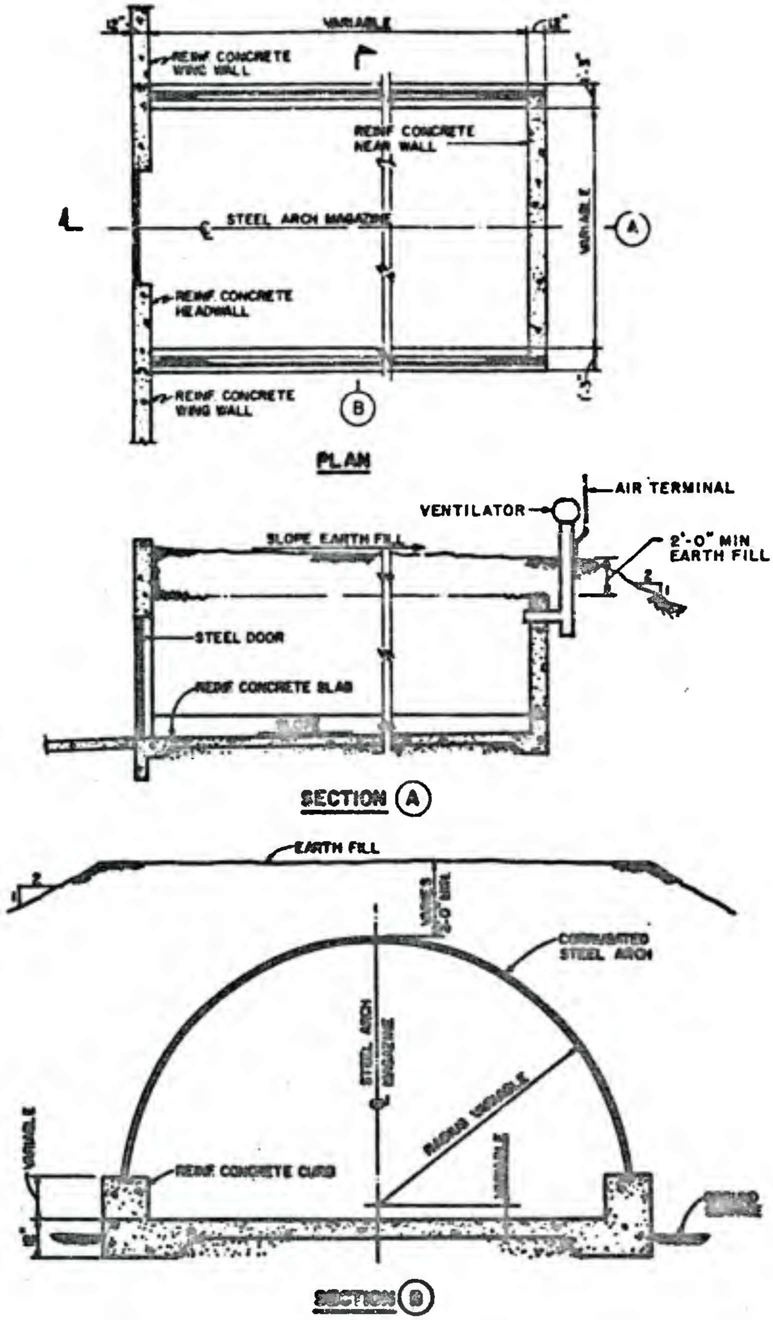
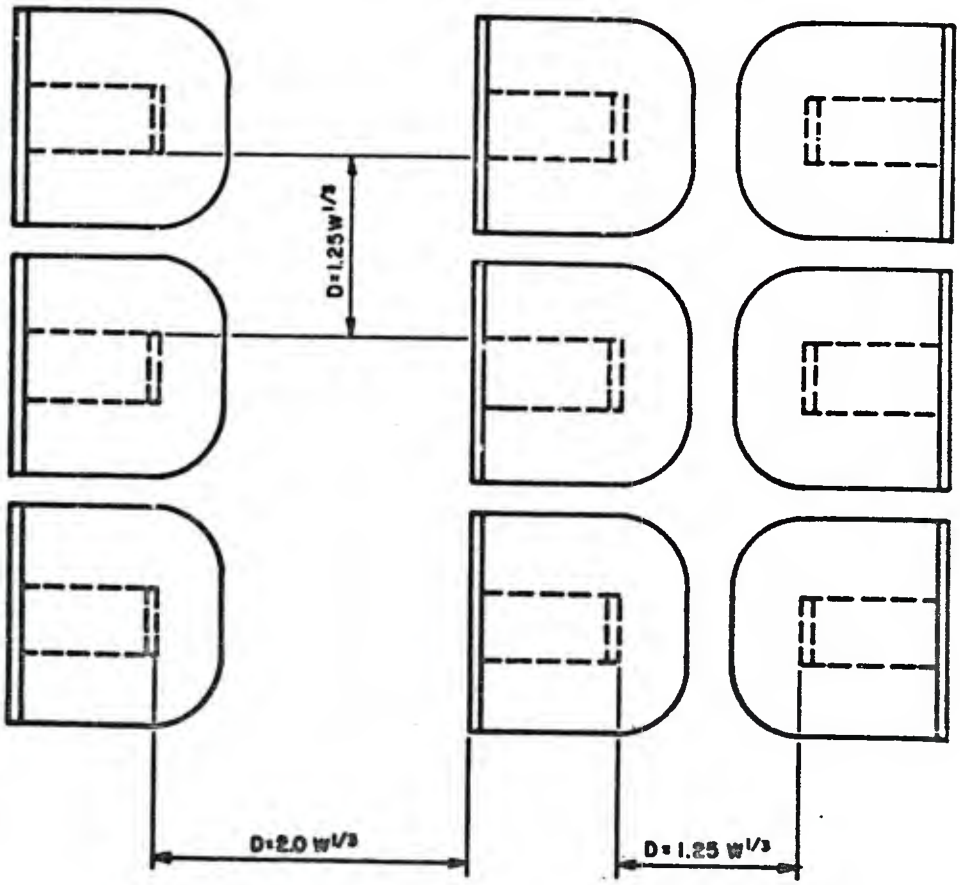


Figure 6-48 Typical earth-covered steel-arch magazine



═══	HEADWALL
=====	REAR WALL
———	EDGE OF EARTH COVER
– – –	EDGE OF ARCH
W =	CHARGE WEIGHT

Figure 6-49 Minimum separation distances of standard magazines

Design of presently used magazines is essentially conventional except for two features, which are doors and arch. The doors are designed to withstand the dynamic forces from an explosion in a nearby magazine if the siting is in accordance with figure 6-48. However, they provide almost no resistance to the effects of an explosion within the magazine. Also, the capacity of the doors to resist elastic rebound and negative phase pressures may be less than their capacity to resist positive phase pressures. Therefore, where personnel are concerned, all doors should be analyzed to determine their ultimate capacity to resist all the loadings involved. The arches used for the standard earth-covered magazines are the same as those used in the test structures to establish the required separation distances. These arches have not been dynamically designed for the blast loads and may be in excess of that required.

6-39 Construction

Effectiveness of earth-covered magazines is largely determined by the quality of construction. A few of the construction details that could be sources for problems in this type of structure are discussed below.

Moisture proofing of any earth-covered structure is usually difficult. This difficulty is increased with a steel-arch structure because of the many lineal feet of joints available for introducing moisture. For example, a large 26- by 80-foot magazine contains approximately 1,030 feet of edges. A sealant tape must be used that will not deteriorate or excessively deform under any anticipated environmental or structural condition.

Earthfill material should be clean, cohesive, and free from large stones. A minimum earth cover of two feet must be maintained. Surface preparation of the fill is usually required to prevent erosion of the 2-foot cover.

Restricting granular size of material reduces throwout of fragments in case of an accidental explosion and creates a more uniform energy absorbent over the top of the magazine.

Lightning protection is rather easily obtained in a steel-arch structure. All sections of steel-arch plate must be interconnected so that they become electrically continuous. In a concrete-arch magazine, the reinforcing steel must be interconnected. In effect, a "cage" is created about the magazine contents. Probably the most critical point for lightning protection is the optional ventilator stack which projects above the surrounding earth cover.

6-40 Non-Standard Magazines

Non-standard earth-covered magazines, that is magazines not listed in DoD Standard 6055.9-STD may also be used for explosive storage. However, if a "non-standard" earth-covered magazine or an aboveground magazine is used the separation distances must be increased. The DoD Standard 6055.9-STD includes the increased separation distances, as well as other criteria, for "non-standard" earth-covered and aboveground magazines.

BLAST VALVES

6-41 General

6-41.1 *Applications*

A prime concern of blast resistant structures is to restrict the flow of high pressures into or out of a structure. For donor structures pressures released may have to be restricted in order to limit pressures acting on adjacent structures to tolerable levels. Also, pressures leaking into acceptor structures must be limited to prevent pressure buildup beyond acceptable levels. In either case openings may have to be completely sealed to prevent the diffusion of contaminants.

The simplest, most economical way of limiting leakage pressure into or out of a protective structure is to restrict the number and size of air intake and exhaust openings. In a donor structure the leakage pressures may be further reduced by venting them through a stack. The stack increases the distance from the point of release of the pressure to the acceptor structure thereby attenuating the blast loading. Methods for predicting the pressures leaking out of a building and the pressure buildup within a building are discussed in Volume II. If the leakage pressure can not be reduced to acceptable levels or if contaminants are released during an explosion, the openings must be sealed with either blast valves or other protective closure device.

Blast valves may be either remote-actuated (closed mechanically by remote sensors) or blast-actuated (closed by the pressure wave itself). Both types can be non-latching or latching. A non-latching valve will open under negative pressures. A latching valve is one that can only be reopened manually. In addition, a blast-actuated valve can be double-acting. A double-acting valve will seal against the positive blast pressure, move in the opposite direction to seal against negative pressures and then reopen when pressures return to normal.

6-41.2 *Remote-Actuated Valves*

Remote-actuated valves are dependent on external sensing devices which trigger the closure mechanism and close the valve before arrival of the blast wave. Actuating devices have been developed that are sensitive to the blast pressure of an explosion and react electrically to trigger protective closure systems. Other actuating devices sensitive to flash and thermal radiation are also available. The pressure sensing device is placed on a circumference at a predetermined radius from the valve (closer to ground zero) in order to compensate for time delays of electrical and mechanical control equipment and to permit valve closure before the blast arrives. Thermal sensors are designed to detect the characteristic pulse emitted by an explosion to prevent actuation by other sources such as lightning, fires, etc., which may occur with flash sensors.

Remote-actuated valves present problems of protection against multibursts and button-up time for combustion-type equipment installed within the structure. In addition to problems of hardenability of the exposed sensor and suitability for multiburst operation, it is often necessary for sensors to initiate reopening of the valves as soon as dangerous pressures have subsided. In general, remote-actuated valves can not close fast enough to be effective during an H. E. explosion.

6-41.3 *Blast-Actuated Valves*

Self-acting blast-actuated valves, which close under the action of the blast pressure, overcome some of the disadvantages described above and present other factors to be considered. Since the valves are closed by pressure, they are not dependent upon sensing devices for operation. They can be automatically reopened after passage of the positive phase or latched closed during the negative phase if this is required. Double-acting valves automatically seal the opening during the negative phase. Since blast-actuated valves are closed by the blast, there is an inherent leakage problem to be considered due to the finite closing time. Although this is in the order of milliseconds for most valves, sufficient flow to cause damage may pass the port openings for certain valve designs and pressure levels. Effectiveness of closing at both high- and low-pressure ranges must be checked. The amount of blast entering depends on the closing time of the valve which, in turn, depends on the mass of the moving parts, disk diameter, and the distribution of pressures on both faces of the disk.

Ideally, a blast-actuated valve should possess the following characteristics: instantaneous closure or no leakage beyond the valve during and after closure, no rebound of moving parts, equal efficiency at all pressures below the design pressure, operational and structural reliability, minimum of moving parts, low-pressure drop through the valve at normal ventilation or combustion air flows, multiple detonation capability, durability, and easy maintenance.

Although instantaneous closure is not physically possible, the closing time can be reduced sufficiently to reduce the leakage to insignificant values. This may be accomplished by increasing the activating pressure-force-to-moving-mass ratio, decreasing the length of travel, permitting no deceleration during closure, and other methods.

6-41.4 *Plenums*

Blast valves, especially blast-actuated valves, will allow some pressure leakage. While the leakage pressure may not significantly increase the ambient pressure at some distance from the valve, there might be a "jetting effect" causing very high pressures in the immediate vicinity. A plenum may be used to protect against these high pressures. Two examples of plenums are, a plenum chamber and a plenum constructed of hardened duct work. A plenum chamber is a room where pressures attenuate by expansion. A hardened duct work plenum reduces the pressures by increasing the distance traveled (similar to the stack discussed in Section 6-41.1). For a donor structure, a plenum would only be necessary if contaminants are released during an explosion and

the air must be filtered before being vented to the exterior. In that case, a plenum would be used to lower the pressures and prevent damage to the filters. A plenum in an acceptor structure would be used to prevent high leakage pressures from directly entering the building's air duct system and possibly causing local failure of the system.

Plenum chambers should be designed to avoid a buildup of interior pressure which would impede closing of the valve. The ratio of the area of the chamber cross section to that of the valve outlet should be preferably greater than 4:1 so as to diffuse leakage flow as quickly as possible. A chamber which has the prescribed necessary volume but has little change in area would act like a tunnel wherein entering pressures would encounter little attenuation in the length provided.

6-41.5 *Fragment Protection*

To ensure that blast valves function properly, they must be protected from fragments that may perforate the valves or jam them in an open position. One of several methods of accomplishing the required protection is by offsetting the opening from the blast valve by means of a blast-resistant duct or tunnel which would prevent the propagation of fragments to the blast valve. Another method is to enclose the blast valve in a concrete chimney. Other methods include using a debris pit or steel shields or debris cover attached directly to the blast valves.

6-42 Types of Blast Valves

6-42.1 *General*

Various types of blast valves have been developed and many of them are available commercially from suppliers both here and overseas (see table 6-10). For present designs, air flow rates from about 300 cfm to 35,000 cfm can be obtained. Some valve designs are available in more than one size and can be either blast or remote-actuated. The pressure loss across the valve at the rated air flow is, in most cases, less than one inch of water.

The maximum incident pressure capability of available valves is above 50 psi and generally at least 100 psi. For shelter purposes, these valves may be overdesigned since the protection level for many shelters will be less than 50 psi. Except for cost factors, using a 100-psi valve for a 10-psi shelter design should not necessarily present technical problems since a valve must operate at all pressures below the maximum design level.
The best type of actuation (blast or remote) depends partly on the design pressure as previously discussed with regard to reaction time and operational considerations. For long arrival times (low pressure), a remote-actuated device can close the valve before the blast arrival whereas leakage may occur for a blast-actuated valve. At a high pressure (short arrival time),the closing time for the remote-actuated valve may be longer than the arrival time.

Table 6-10 Blast Valves

Name of Valve		Type (Actuated)	Blast Characteristics		Locking Mechanisms	Air Flow Rates (cfm)	Tested
			Pressure (psi)	Closing Time (ms)			
U.S. Army Chemical Corps M-1		Blast	100	*	Yes	300	Field
U.S. Army Chemical Corps E191R1		Blast	50	*		600	Shock tube and field
Office of Civil Defense	12"ø	Blast and remote	100	*	Yes	600	Field
	16"ø		50	50	Yes	1,200	
	24"ø		50	*	Yes	2,500	
Office of Corps of Engineers		Blast and remote	*	20	Yes	5,000	Shock tube
Bureau of Yards and Docks		Remote	*	500	Yes	2,200 to 30,000	*
Temet USA, Inc.		Blast	100	20	Yes	2,250 to 3,700	Shock tube
Jaern and Plat	4"ø		280	*		150	Field
	8"ø					600	
	14"ø					1,750	
Technical Facilities WS-107 A2		Remote	*	*		35,000	*
American Machine Foundry		Blast	100	*	Yes	80	*
Luwa		Blast	15 to 160	2	No	450	Shock tube
Suffield Experimental Stations		Blast	4 to 20	*		*	*
New Naval Civil Engineering Lab	Brechenridge	Blast	100	*	No	600	Compressed air
	Bayles-Denny		5 to 100	*	No	180	
	Stephenson		5 to 104	*	No	125	
Artos Machinery Co. (Sand Filtered)		Blast	100	*	No	300	Field

*Unknown

6-42.2 *Blast Shield*

If it can be assumed that the ventilation system can be closed off during a hazardous operation and kept closed until there is no danger of further blast, a relatively simple structural closure (blast shield), such as a steel plate can be utilized. This type of closure is especially useful for an exterior opening which would only be opened periodically, such as maintenance facilities where the release of toxic fumes from within the structure is required.

6-42.3 *Sand Filter*

In shelters where normal operational air requirements are small, sand filters are useful in the attentuation of leakage pressures. With this type of filter the pressures continue to increase throughout the positive phase. Thus this filter is good only for loads with a relatively short duration. A sketch of a 300-cfm sand filter is shown in figure 6-50.

6-42.4 *Blast Resistant Louvers*

The blast resistant louver shown in figure 6-51 is blast actuated and has a rated flow of 600 cfm at less than 1 inch water (gage) pressure drop. If a larger volume of air is required the louvers can be set into a frame and used in series (see fig. 6-52). Louvers can be used in acceptor structures subject up to 50 psi. A major drawback of the louvers is that there may be as much as 40 percent leakage across the valve, especially at lower pressures.

6-42.5 *Poppet Valves*

6-42.5.1 *Applications*. A poppet valve has many advantages. It has few moving parts which might need repair. It can be blast or remote-actuated, latching, non-latching, or double-acting and is available in sizes from 600 to 5,000 cfm. A blast-actuated poppet valve has a very fast closing time, approximately 20 milliseconds, making it the only valve that reacts fast enough to be used in a containment cell.

A typical blast-actuated poppet valve is shown in figure 6-53. The valve consists of an actuating plate, a valve seat, a backing plate that supports the actuating plate, and a spring which holds the valve open during normal operations. The normal air flow is around the actuating plate. A blast load will compress the spring and move the actuating plate against the valve seat thereby sealing the opening. As the blast pressure moves the actuating plate, some pressure will flow around the plate while it is closing.

The leakage pressures can be completely prevented by using a valve similar to the one schematically illustrated in figure 6-54. In this valve the normal air flow is around the actuating plate through a length of duct. When the valve is subjected to a blast load, the pressure starts moving the actuating plate while at the same time flowing through the duct. The length of the duct must be long enough to ensure that the time it takes the blast pressures to flow through the duct (delay path) will be longer than the time required for the actuating plate to seal the valve. As an alternate to the long duct, an expansion chamber may be used to delay the blast.

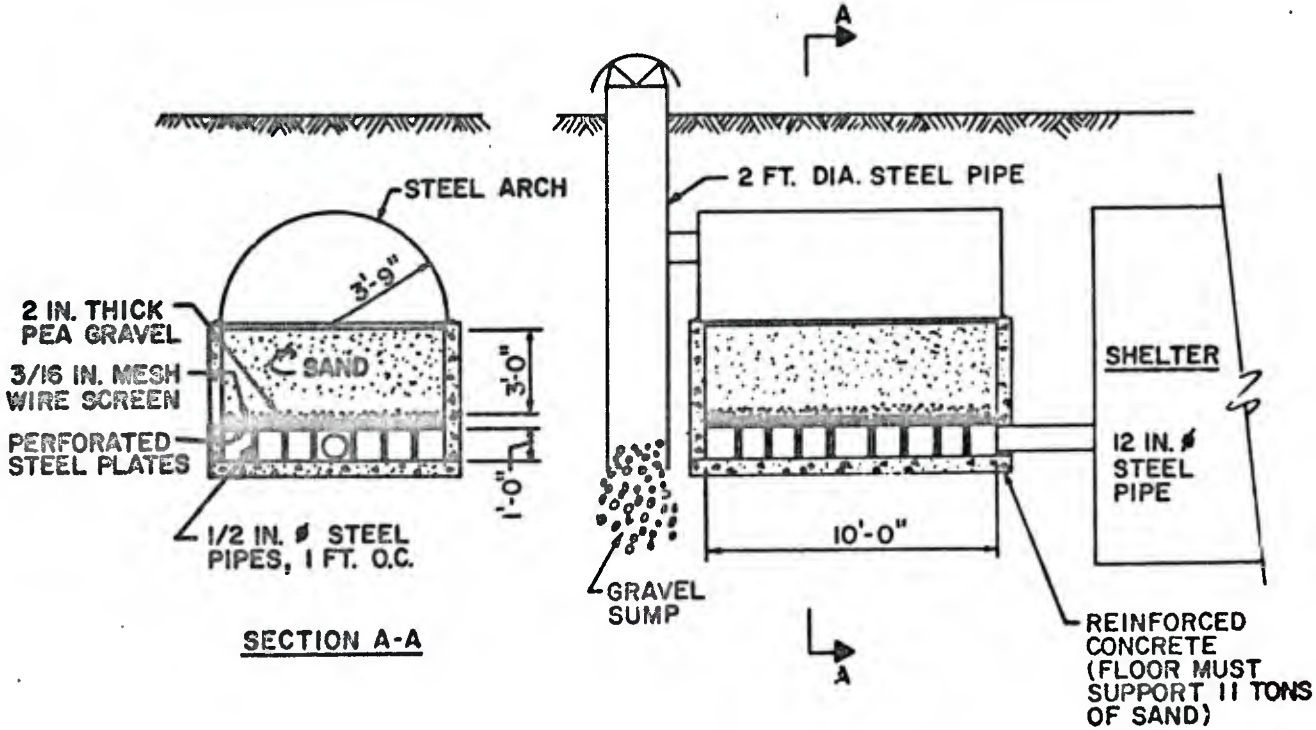


Figure 6-50 Sketch of 300 cfm sand filter

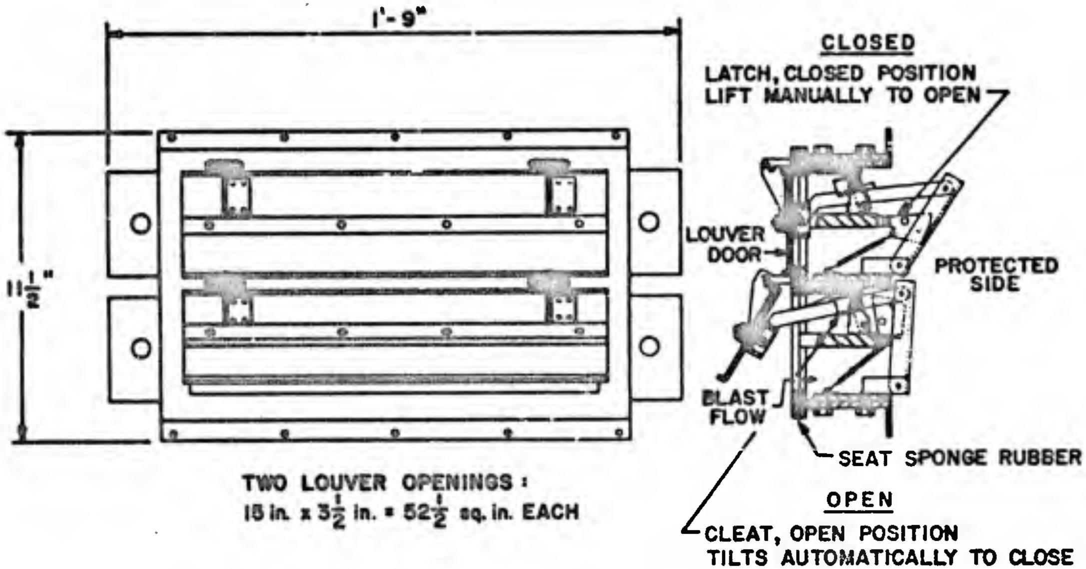


Figure 6-51 Blast-actuated louver

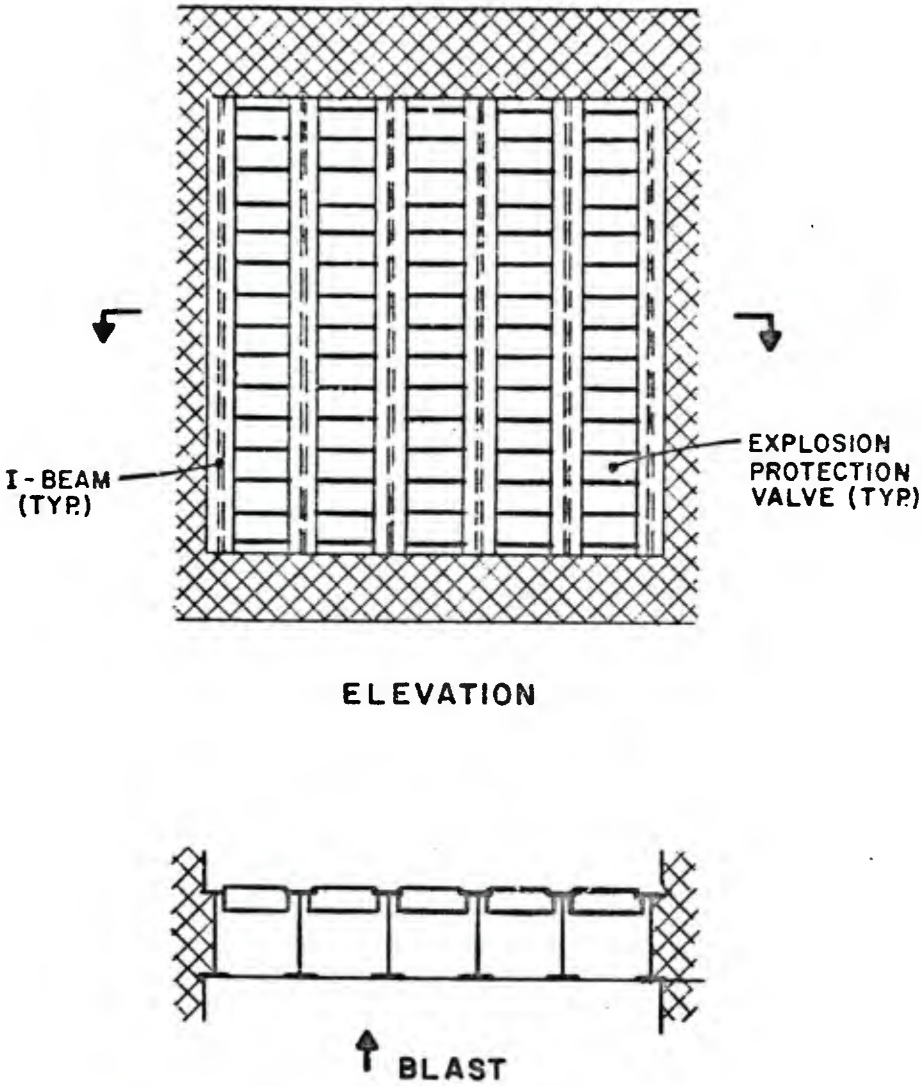


Figure 6-52 Arrangement of multiple louvers for a large volume of air

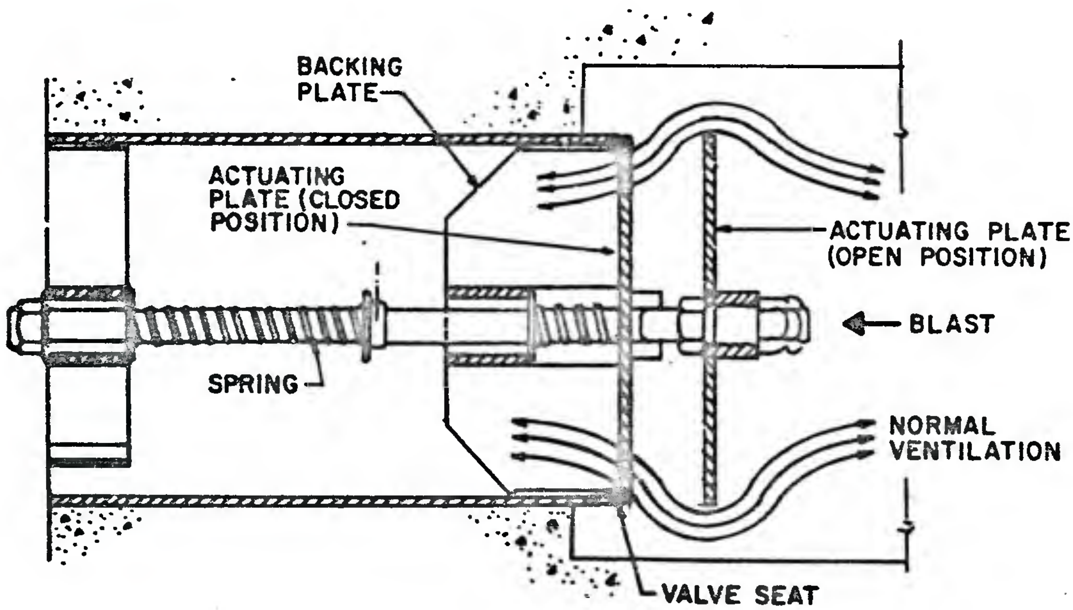


Figure 6-53 Typical blast-actuated poppet valve

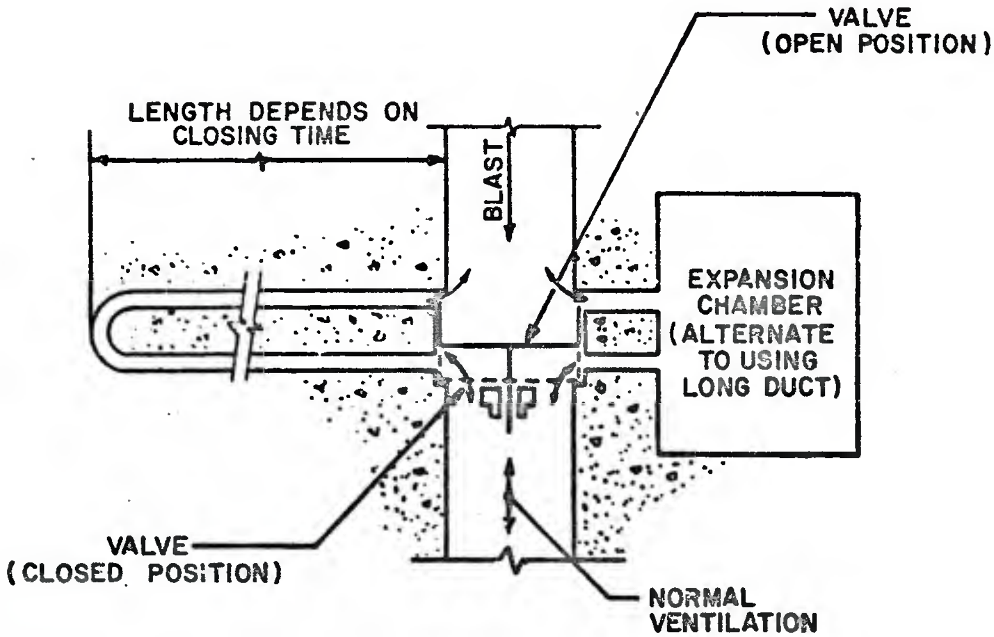


Figure 6-54 Time delay path

6-42.5.2 *Recommended Specification for Poppet Valves.* Presented below is an example specification for the design, testing and construction of a poppet valve, but it may be adapted for other types of blast valves. This example specification is presented using the Construction Specification Institute (CSI) format and shall contain as a minimum the following:

1. APPLICABLE PUBLICATIONS: Except as otherwise stated herein all materials and work furnished in accordance with this specification shall comply with the following codes and standards.

1.1 Federal Specification (Fed. Spec.)

TT-P-37D	Paint, Alkyd Resin; Exterior Trim, Deep Colors
TT-P-645A	Primer, Paint Zinc Chromate Alkyd Type
TT-P-86G	Paint, Red- Lead-Base, Ready Mixed

1.2 American Society for Testing and Materials (ASTM)

A 53-81a	Pipe, Steel Black, and Hot-Dipped, Zinc-Coated, Welded and Seamless
D 2000-80	Rubber Products in Automotive Applications, Classification System For
E-709-80	Magnetic Particle Examination

1.3 American Institute of Steel Construction (AISC)

Specification for the Design, Fabrication and Erection of Structural Steel for Buildings (Eighth Edition) with commentary

1.4 American Iron and Steel Institute (AISI)
304 17-7th

1.5 American Welding Society (AWS)
D.1.1 Structural Welding Code (latest edition)

2. SUBMITTALS: The following information shall be submitted for approval. Materials shall not be delivered to the site until approved shop drawings have been returned to the Contractor. Partial submittals, or submittals for less than the whole of any system made up of interdependent components will not be accepted. Submittals for manufactured items shall be manufacturer's descriptive literature, shop drawings, and catalog cuts that include the manufacturer's dimensions, capacity, specification reference, and all other information necessary to establish contract compliance.

2.1 Qualifications: The Contractor shall submit for approval, data to support the qualifications of the manufacturer and installer. A list of previously successfully completed jobs of a similar nature, indicating the name and address of the owner of the installation shall be included with the information.

2.2 Manufacturer's Data: Before executing any fabrication work, a completely marked and coordinated package of documents sufficient to assure full compliance with the drawings and specifications shall be submitted. The submittal shall include a complete technical evaluation of the capacity of each valve as described below. The drawings shall include: detailed fabrication and equipment drawings; assembly showing the complete installation, including methods for supporting the valves, subframes and frames; a listing of all materials and material specifications; surface finishes; fabrication, assembly and installation tolerances; locking devices and locking device release mechanisms; and a detailed sequence for installation of valves and frames in conjunction with other phases of construction. Structural fabrication drawings shall conform to the requirements of AISC and welding shall conform to AWS. Assembly and installation shall be based on field established conditions and shall be fully coordinated with architectural, structural, and mechanical systems. All aspects of any work developed in connection with the development of these valves shall be fully documented and become the property of the Government.

2.2.1 Standard Compliance: Where equipment or materials are specified to conform to requirements of the standards of organizations such as ANSI, NFPA, UL, etc., which use a label or listing as a method of indicating compliance, proof of such conformance shall be submitted for approval. The label or listing of the specified organization will be acceptable evidence. In lieu of the label or listing, the Contractor shall submit a notarized certificate from a nationally recognized testing organization adequately equipped and competent to perform such services, and approved by the Contracting Officer stating that the items have been tested with the specified organization's methods and that the item conforms to the specified organization's standards.

2.3 Preliminary Hydraulic Characteristics: Prior to Construction, submit with shop drawings an estimate of the hydraulic characteristics of each valve, under actual operating conditions. Ratings shall be based on tests or test data. All necessary corrections and adjustments shall be clearly identified. Corrections shall be established for actual altitude and air flow directions as shown on the drawings as well as hydraulic effects produced by mounting and/or connection of the valve.

2.4 Tests and Test Reports: Except as noted otherwise, the testing requirements for materials stated herein or incorporated in referenced documents, will be waived, provided certified copies of reports of tests from approved laboratories performed on previously manufactured materials are submitted and approved. Test reports shall be accompanied by notarized certificates from the manufacturer certifying that the previously tested material is of the same type, quality and manufacture as that furnished for this project.

2.5 Blast pressure analysis calculations and/or results of approved tests shall be submitted, for both the blast valve and subframe, for approval and shall conform to the requirement of the paragraph entitled, BLAST VALVE TESTS. The calculations and/or test results shall include all components of the valves and subframe subjected to the blast pressures. Calculations are not required for the frame embedded in the concrete. This frame shall conform to that shown on the drawings. However, the fabrications of the blast valve and associated subframe and embedded frame shall be the responsibility of the blast valve manufacturer. This manufacturer shall also be responsible for installation of this equipment.

2.6 Qualification of Welders: Before assigning any welder to work covered by this section of the specification, the Constractor shall submit the names of the welders to be employed on the work together with certification that each of these welders has passed the qualification test using procedures covered in AWS Standard D1.1.

2.7 Operational and Maintenance Manual: Operation and maintenance manuals shall be furnished by the Contractor. Complete manuals shall be furnished prior to the time of installation. The manual shall have a table of contents and shall be assembled to conform to the table of contents with tab sheets placed before instructions covering the subject.

2.8 Shop Test Reports: The Contractor shall furnish copies of shop inspection and test results of fabrication welding.

3. MATERIALS:

3.1 Structural steel pipe used for blast valve construction shall conform to ASTM A 53 seamless pipe.

3.2 All structural steel plate components of the valve shall consist of stainless steel and shall conform to AISI 304.

3.3 Spring type components shall consist of stainless steel and conform to AISI 17-7th, Condition C.

3.4 Blast Seal Material: Seals for blast valves shall conform to ASTM D 2000.

4. BLAST VALVE REQUIREMENTS: All blast valves shall be poppet type and shall have the following characteristics.

4.1 Pressure capacity: Each valve shall be capable of withstanding a sustained blast pressure of 100 psi as well as the impact force produced by the closing of the valve. The valve shall be designed to sustain elastic deformation when subjected to the above loads. The blast valve shall be capable of closing under a force of 15 pounds.

4.2 Temperature Capacity: Each valve shall be capable of satisfactory operation over a temperature range of 35° to 300° F.

4.3 Valve Actuation: Each valve shall be actuated by the blast overpressure. The valve shall be in the closed position 20 milliseconds after the onset of the blast front. The blast pressures are given on the drawings.

4.4 Valve Parameters: A minimum of 12-inch diameter blast valve shall be used. After the valve is closed by the blast overpressure, it shall remain in the closed position until manually opened. This shall require that the valve be equipped with a locking device which shall be located at the exterior side of the valve. A release mechanism for the locking devices shall be provided which shall be operated from a position immediately adjacent to the interior of the valve. Any penetration through the valve or the structure must be capable of being sealed against blast leakage through the penetration. The air flow capacity of the valves shall be 1500 SCFM (1710 ACFM) for the supply and return valves and 880 SCFM (1000 ACFM) for the exhaust valve. Total actual pressure drop across the valve with air movement in either direction shall not exceed one inch of water (gage). The valve and its operating parts shall be designed for a 20-year life and shall have an operating frequency of 10,000.

4.5 Blast Seals: Blast seals shall be provided between the face of each valve and subframe and between the subframe and the frame to provide a pressure tight condition. Seals shall be adjustable and easily replaceable. The seal shall be designed to be leakproof with a pressure differential across the seal of 100 psi.

4.5.1 Blast Seal Material: Seal material shall conform to ASTM D 2000. Four sets of blast seals shall be furnished with each valve. Three sets of the seals shall be packaged for long term storage.

4.5.2 Adhesive: Adhesive for blast seals shall be as recommended by the manufacturer of the seals. Sufficient adhesive shall be provided for installation of the packaged seals at a later date.

4.6 Field Removal: Blast valves shall have the capabilities of being completely field removed and disassembled.

5. FABRICATION:

5.1 Qualification of Manufacturer: The manufacture and installation of blast valves and frames shall be performed by the blast valve manufacturer who shall be fully responsible for valve operation. The manufacturer shall have complete facilities, equipment and technical personnel for the design, fabrication, installation and testing of complete blast valve assemblies.

5.2 General: The drawings indicate the location of the blast valves in the structure. The manufacturer shall carefully investigate the drawings and finished conditions affecting his work and shall design the units to meet the job condition and the dynamic loads. The blast valves shall be complete with gaskets, fasteners, anchors, mechanical operators, and all other equipment and accessories as required for complete installation.

5.3 Metalwork: Except as modified herein, fabrication shall be at a minimum, in accordance with the AISC Specification for the Design, Fabrication and Erection of Structural Steel for Buildings. Welding of steel shall be in accordance with the requirements for AWS Specification D1.1. A welding sequence to reduce distortion and locked-up stresses to a minimum shall be used. All welded units shall be stress relieved. All welded members shall be post weld straightened free of twist and wind. Fabricated steel shall be well formed to shape and size, with sharp lines and angles. Exposed welds shall be ground smooth. Exposed surface of work, in place, shall have a smooth finish. Where tight fits are required joints shall be milled to a close fit. Corner joint shall be coped or mitered, well formed, and in true alignment. Permanent connections for all assemblies and components, except those requiring removal for installation and maintenance, shall be welded. Each valve and subframe shall be removable from the embedded frame.

5.3.1 Machining: Parts and assemblies shall be machine finished wherever necessary to insure proper fit of the parts and the satisfactory performance of the valves.

5.3.2 Weld Details: The types of edge preparation used for welds shall be chosen by the manufacturer to be the most suitable for the joint and position of welding. Where required, all groove welds shall be complete penetration welds with complete joint fusion. Groove weld edge preparations shall be accurately and neatly made. All full penetration groove joints shall be back-chipped and back welded where both sides are accessible. Where both sides are not accessible, backing strips not exposed to view may be left in place unless removal is required for clearance. Backing strip not removed shall be made continuous by welding ends and junctions.

5.3.3 Weld Tests: Inspection and tests of welds shall be as specified in AWS Specification D1.1. All welding shall be subjected to normal continouous inspection.

5.3.3.1 Nondestructive dye penetrator testing shall be performed for all welding in accordance with Method B of ASTM E 165 or ASTM E 709. Allowable defects shall conform to AWS Specification D1.1.

5.3.3.2 Penetration Welds: All full or partial penetration corners, tees and inaccessible butt joints shall be subjected to 100 percent ultrasonic examination. All penetration joints shall be considered to be

tension joints. All tests shall be performed by a testing laboratory approved by the Contracting Officer. The testing laboratory shall be responsible for interpretation of the testing, which shall be certified and submitted in a written report for each test. In addition to the weld examination performed by the Contractor, the Contracting Officer reserves the right to perform independent examination of any welds at any time. The cost of all Government reexamination will be borne by the Government.

5.3.3.3 Correction of Defective Welds: Welds containing defects exceeding the allowable which have been revealed by the above testing shall be chipped or ground out for full depth and rewelded. This correction of the defected weld area and retest shall be at the Contractor's expense.

5.4 Metal Cleaning and Painting:

5.4.1 Cleaning: Except as modified herein, surfaces shall be cleaned to bare metal by an approved blasting process. Any surface that may be damaged by blasting shall be cleaned to bare metal by powered wire brushing or other mechanical means. Cleaned surfaces which become contaminated with rust, dirt, oil, grease, or other contaminants shall be washed with solvents until thoroughly clean.

5.4.2 Pretreatment: Except as modified herein, immediately after cleaning, steel surfaces shall be given a crystalline phosphate base coating; the phosphate base coating shall be applied only to blast cleaned, bare metal surfaces.

5.4.3 Priming: Treated surfaces shall be primed as soon as practicable after the pretreatment coating has dried. Except as modified herein, the primer shall be a coat of zinc chromate primer conforming to Fed. Spec. TT-P-645, or a coat of red lead paint, Type I or Type [illegible] conforming to Fed. Spec. TT-P-86G, applied to a minimum dry film thickness of 1.0 mil. Surfaces that will be concealed afte[illegible] construction and will require no overpainting for appearance may be primed with a coat of asphalt varnish, applied to a minimum dry film thickness of [illegible].0 mil. Damage to primed surfaces shall be repaired with the primer.

5.4.4 Painting: Shop painting shall be provided for all metalwork, except for non-ferrous metals and corrosion resistar[illegible] metals and surfaces to be embedded in concrete. Surfaces to be welded [illegible] not be coated within three inches of the weld, prior to welding. All machined surfaces in contact with outer surfaces and bearing surfaces shall not be painted. These surfaces shall be corrosion protected by the application of a corrosion preventive compound. Surfaces to receive adhesives for gaskets shall not be painted. Surfaces shall be thoroughly dry and clean when the pai[illegible] is applied. No painting shall be done in freezing or wet weather except under cover; the temperature shall be above 45° F but not over 90° F. Paint shall be applied in a workmanlike manner and all joints and crevices shall be coated thoroughly. Surfaces which will be concealed or inaccessible after assembly shall be painted prior to assembly. Paint shall conform to Fed. Spec. TT-P-37D.

6. BLAST VALVE TESTS:

6.1 Response Tests: The following static and dynamic response shall be performed to demonstrate the blast resistant capabilities of the blast value design. These tests shall be witnessed by the Contracting Officer.

6.1.1 Closure Time Test: Prior to shipment to the site, the Contractor shall perform a test to demonstrate that the closure of the blast valve will not exceed the 20 milliseconds specified. A suggested method for recording the valve closure is with the use of a high speed camera.

6.1.2 Static Pressure Test: Prior to shipment to the site, the Contractor shall perform a pneumatic test to demonstrate the static capacity of the blast valve design. The valve must sustain the pressure of 100 psi for a minimum of two hours. The total pressure loss during that period shall not exceed 1 psi.

6.1.3 Dynamic Pressure Test: Prior to shipment to the site, the Contractor shall perform a test to demonstrate the dynamic capacity of the blast valve design. This test shall simulate the combined effects of impact forces produced by the valve closure system and the blast load. This test may be replaced by design analyses which demonstrate that the head and frame of the valve shall have the capability to resist the stresses produced by the above forces. The blast load as indicated on the drawings shall be used for this analysis.

6.1.4 Blast Tests: If the effects of one or more of the above blast valve tests have been demonstrated by prior blast valve tests on similar valves, then the results of these tests shall be submitted for review; and the above test performances may not be required.

6.2 Test: After installation, a trip test shall be performed and demonstrated to the operating personnel.

6.3 Hydraulic Characteristics: Prior to shipment to the site, the Contractor shall perform a final test to establish the hydraulic characteristics of each valve and provide the necessary corrections and adjustments as stated previously.

SHOCK ISOLATION SYSTEMS

6-43 Introduction

Previous sections have presented methods for the prediction of blast and fragment effects associated with the detonation of H. E. explosives and the design or analysis of structures to withstand these effects. In the design of shelters, an important part of the design process is to insure the survival of personnel and equipment. It is possible that the structure could withstand the air blast and ground shock effects but the contents be so severely damaged by structural motions that the facility could not accomplish its intended function. A similar problem is in the design of shelter type structures that

houses sensitive explosives. These explosives must be protected from structure motions since these motions could result in initiation of the explosive. This section deals with the protection of vulnerable components from structure motions due to air blast and ground shock.

6-44 Objectives

The objective of shock isolation in protective design applications is to reduce the magnitude of motions transmitted by a vibrating structure to personnel or shock sensitive equipment. These motions must be attenuated to levels tolerable to personnel and to be various pieces of equipment used in the facility. A second consideration in some cases is to reduce the magnitude of motions transmitted by vibrating equipment to its supports. These latter motions can be significant for equipment mounted on shock isolated platforms.

The general functional objectives of a shock isolation system are:

1. Reduce input motions to acceptable levels.
2. Minimize rattle space requirements consistent with system effectiveness and cost.
3. Minimize coupling of horizontal and verticle motions.
4. Accommodate a spectrum of inputs of uncertain waveforms.
5. Limit the number of cycles of motion of the isolated body.
6. Support the system under normal operating conditions without objectionable motions.
7. Maintain constant attitude under normal operating conditions.
8. Accomodate changes in load and load distribution.
9. Maintain system vibration characteristics over long periods of time.
10. Interface properly with other components or parts.
11. Require minimum maintenance

6-45 Structure Motions

Ground shock results from the energy which is imparted to the ground by an explosion. Some of this energy is transmitted through the air in the form of air-blast-induced ground shock and some is transmitted through the ground as direct-induced ground shock. Both of these forms of ground shock when imparted to a structure will cause the structure to move in both a vertical

and horizontal direction. Movement of the structure imparts motions to items attached to the structure's interior. Motion of interior items is obtained from a response spectrum. This is a plot giving the maximum responses (in terms of displacement, velocity, and acceleration) of all possible linear single-degree-of-freedom systems which may be attached to the structure due to a given input motion. Therefore, having the spectra for the structure and given input motion, the maximum response of any item within the structure is obtained based on the natural frequency of the item. Methods for preparing response shock spectra is presented in Volume II of this manual.

In addition to motion of the structure as a whole, the exterior walls and roof respond to the direct application of the blast load. Methods for calculating the response of these elements are given in Volume III of this manual using the parameters given in Volume IV and V for concrete and steel, respectively. Maximum displacements, velocities, and accelerations of these elements can be determined in a straightforward manner. These quantities can be used to determine effects on items attached or located near walls or roofs.

6-46 Shock Tolerance of Personnel and Equipment

The requirement for shock isolation is based upon the shock tolerance of personnel and/or critical items of equipment contained within the protective structure. If the predicted shock input exceeds the shock tolerance of personnel; a shock isolation system is required. If the shock input exceeds the shock tolerance of equipment, the equipment can either be ruggedized to increase its shock tolerance or it can be shock isolated. There are practical limits to ruggedization and the costs may exceed those of an isolation system. If the input does not exceed the shock tolerance of the equipment, it can be hard-mounted to the structure.

6-46.1 *Personnel*

The effects of structural motions on personnel depend on the magnitude, duration, frequency, and direction of the motion, as well as their position at the time of the loading. The shock tolerance of personnel is presented in Volume I of this manual.

6-46.2 *Equipment*

In many cases, the need for shock isolation of equipment must be established before detailed characteristics of the system components are established. Further, because of the constraints of procurement procedures, shock isolation systems must be designed and built prior to specific knowledge of the equipment to be installed. In such instances, the choice lies between specifying minimum acceptable shock tolerances for the new equipment or using whatever data is available for similar types of equipment.

The most practical means of determining the shock tolerance of a particular item of equipment is by testing. However, even experimental data can be of questionable value if the test input motion characteristics differ greatly from those that would actually be experienced by the equipment. Since testing

of equipment may not be practical in many cases due to the amount of time allotted from the inception of a project to its completion, procurement procedures, and cost limitations, it is often necessary to rely on data obtained from shock tests of similar items. The shock capacity of various types of equipment is presented in Volume I of this manual.

6-47 Shock Isolation Principles

6-47.1 *General Concepts*

A full treatment of the problem of shock isolation systems is not possible in this manual. The following discussion provides an introduction to the subject and presents some of the important characteristics of shock isolation systems.

In general, the analytical treatment of shock isolation systems is based upon the principles of dynamic analysis presented in Volume III. In most cases, the actual system can be represented by a simplified mathematical model consisting of a rigid mass connected by a spring and dash pot as shown in figure 6-55. The figure represents the simplest case, that of a single-degree-of-freedom system restrained to move in only one direction. Actually, an isolation system would have at least six degrees of freedom, i.e., three displacements and three rotations. Under certain conditions, these six modes can be uncoupled and the system analyzed as six single-degree-of-freedom systems.

The single-degree-of-freedom system shown in figure 6-55 can be used to illustrate the importance of some of the parameters affecting the effectiveness of shock isolation systems in general. The isolator is represented by the linear spring and viscous damping device enclosed within the dotted square. The suspended mass is taken to be a rigid body. It is assumed that the base of the system is subjected to a periodic sinusoidal motion whose frequency is f. The undamped natural frequency of the system is

f_n is given by:

$$f_n = \frac{1}{2\pi} \left(\frac{386.4}{W}\right)^{1/2} \qquad \text{6-55}$$

where

f_n = natural frequency of vibration
K = unit stiffness of spring
W = weight supported by spring

When the frequency of the disturbing motion f is small compared to the natural frequency f_n of the single-degree-of-freedom system, the displacement of the mass is approximately equal to the displacement of the base. When the frequency of the base motion is several times that of the system, the motion of the mass is a small fraction of the base motion. When the ratio of frequencies become large (20 to 30), the system can not respond to the base motion to any significant degree. At frequency ratios near one, large motions of the mass are possible and the magnitude is strongly affected by the amount of damping in the system.

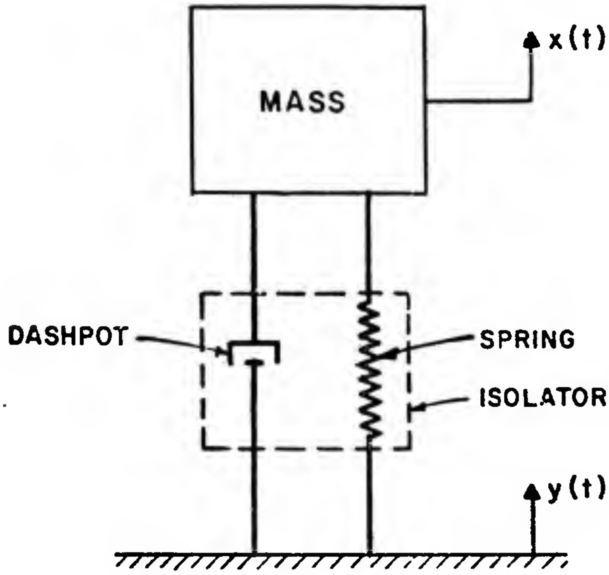


Figure 6-55 Idealized model of shock isolated mass

One obvious shock isolation approach is to use a low frequency suspension system so that the ratio of frequencies is always large. However, low frequency (referred to as soft) systems possess the undesirable characteristic of larger static and dynamic displacements and greater probability of coupling between modes of vibration. Although soft systems may be acceptable under some conditions, the obvious constraint that will preclude their use is a limit on the relative motion between the suspended mass and its supports or adjacent parts of the facility. This relative motion determines the amount of rattle space that must be provided to avoid impact between the mass and other fixed or moving parts of the facility.

The acceleration of the mass is a function of the forces applied to the mass by the spring and damping devices. In the case of a linear undamped spring, the force is a function of the relative displacement between the mass and its support. In viscous damping devices, the damping force is a function of the percent damping and the relative velocity between the mass and its supports. Acceleration limits for the critical items will impose restraints on spring stiffness and the amount of damping in the isolation system. In practice, a compromise combination of spring stiffness and damping is necessary to minimize input motions to the mass for a specified allowable rattle space or to minimize the rattle space required for specified allowable motions of the mass.

The need to avoid resonance (ratio of the frequency of the base motion to the natural frequency of the isolation system equal to one) is obvious. The structural motions resulting from an H. E. explosion are not steady-state sinusoidal in nature. However, these motions are of an oscillatory type and the displacement-frequency relationships discussed above are approximately applicable. A more detailed discussion of the effects of load duration, non-linear springs, damping, and system frequency on response can be obtained from publications listed in the bibliography.

The basic objective in shock isolation is to select a combination of isolation system properties which will reduce the input motions to the desired level. In design, it is a straightforward process. System properties are assumed and an analysis is performed to determine its response to the input motions. If the shock tolerance and rattle space criteria are not satisfied, the system must be altered and the analysis repeated until the criteria are satisfied.

6-47.2 *Single-Mass Dynamic Systems*

A single mass system can have six degrees of freedom, that is, translation in three orthogonal axes and three rotations. The system can also be classified as coupled or uncoupled.

A coupled system is one in which forces or displacements in one mode will affect or cause a response in another mode. For example, a vertical displacement of a single rigid mass might also cause rotation of the mass. An uncoupled system, on the other hand, is one where forces or displacements in one mode do not generate a response in another mode. If the system is completely uncoupled, base translations in any one of the three orthogonal directions will cause translations of the mass in that direction only.

Similarly, a pure rotation of the base about any one of the three orthogonal principal inertia axes with their origin through the mass center, will cause only pure rotations of the body about that axis. The principal inertia axes are those about which the products of inertia vanish. The principal elastic axes of a resilient element (isolator) are those axes for which an unconstrained element will experience a displacement collinear with the direction of the applied force. If the principal elastic axes and the principal inertia axes of the shock isolation system coincide with the origin or point of intersection of both sets of axes at the center of gravity of the mass, the modes of vibration are uncoupled. Such a system is also referred to as a balanced system.

In figure 6-56, if all the springs have the same elastic stiffness, the elastic center will be located at point A, which, in this case, is at the center of the individual springs. If the suspended mass is of uniform density, its center of gravity is also located at A, and the system is uncoupled for motion input through the springs. Some systems may be uncoupled only for motions in a particular direction. If point B in figure 6-56 is the center of gravity of the mass, a horizontal motion in the direction parallel to the X-axis of the structure would cause only a horizontal motion of the mass. A vertical motion of the structure would cause both a vertical and rotational motion of the mass. In this case, the vertical and rotational modes are coupled. If the center of gravity were located at point C, then vertical, horizontal and rotational modes are coupled. If the characteristics of the mass and shock isolation system are such that the modes of vibration can be uncoupled, the system can be analyzed as a series of independent single-degree-of-freedom systems. The response of each of these systems can be determined on the basis of input motions and isolator properties in a direction parallel to or about one of the principal inertia axes. The response in each one of these modes can be summed in various ways to obtain the total response of the system. The sum of the maximum responses would neglect differences in phasing and should represent an upper limit of the actual motions. It is recommended that the square root of the sum of the squares of the maximums (root mean square values) be used to represent a realistic maximum response since it is unlikely that response will occur simultaneously in all modes. Superposition of modal response is appropriate for elastic systems only.

A dynamically balanced shock isolation system offers advantages other than a simplification of the computation effort. A balanced system results in reduced motions during oscillation. As a result of the absence of coupling of modes in a balanced system and the usually small, if any, rotational inputs to the system in protective construction applications, rotational motions of the shock isolated mass will be minimized. This is particularly important for large masses where small angles of rotation can result in large displacements at locations far from the center of gravity.

Because of the advantages of a dynamically balanced system, various approaches are taken to minimize coupling of modes. One criterion is that frequencies in the six modes should be separated sufficiently to avoid resonance between modes. Because of the importance of minimizng rotational modes of response, it is suggested that extremely low stiffnesses in these modes should be avoided.

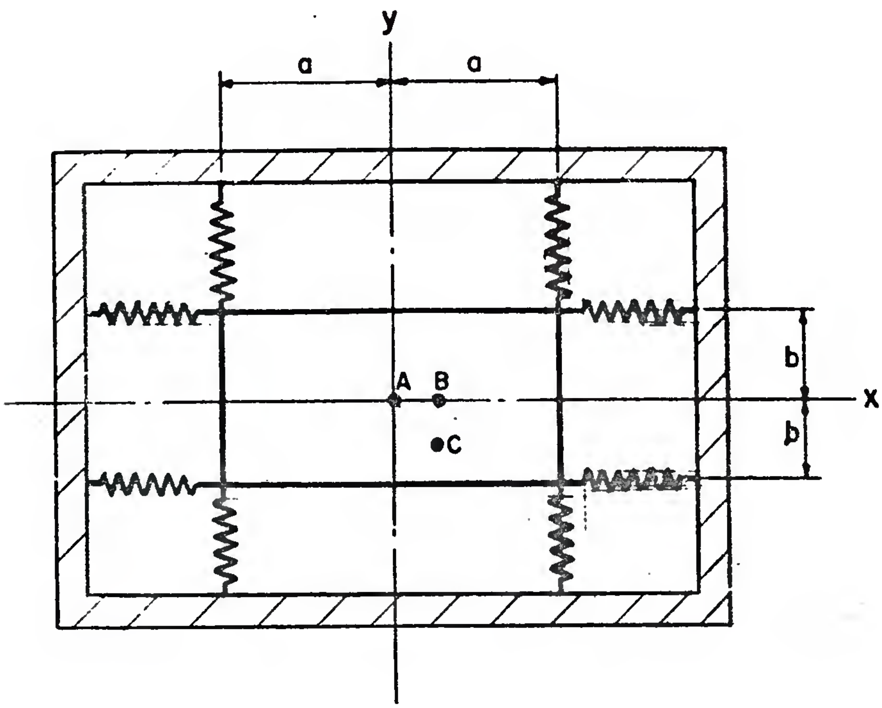


Figure 6-56 Shock isolation system

For the analysis of multiple degrees of freedom, single-mass systems where the various modes of response are coupled, the modal method of analysis or numerical integration techniques can be utilized. The modal method of analysis requires solution of simultaneous equations of motion to determine characteristic shapes and frequencies of each mode and is limited to the elastic case. The numerical techniques do not require prediction of mode shapes and frequencies and will handle both elastic and inelastic response. If the dynamic system is also a multiple mass system, the above methods can be utilized to analyze the system. While an in-depth discussion is beyond the scope of this manual, a complete discussion of these methods can be found in publications listed in the bibliography.

6-47.3 *Shock Isolation Arrangements*

6-47.3.1 *Individual versus Group Mounting.* The two basic approaches to shock isolation in protective construction are to provide individually tailored systems for each component and to group together two or more items on a common platform. In the latter case, the system is selected to satisfy the requirements of the most critical item. In some cases, where the shock tolerance of the various items differs greatly, a combination of the two approaches may be the most effective solution. Although the relative location or size of some items may make individual mounts the more practical approach in certain cases, group mounting will generally be as reliable and the least costly solution.

Where personnel must be protected, a platform is the most practical solution. Except for extremely sensitive equipment, the shock tolerance of the personnel will govern the design of the system. The combination of personnel and equipment on the same platform will permit the personnel to move freely (however cautiously) between items of equipment. Where personnel are not required to be mobile, but rather may be able to remain seated while operating the equipment during hazardous periods, the shock tolerance of the personnel are greatly increased. This increased tolerance will reduce the shock isolation requirements while at the same time affording a higher degree of protection for personnel since they are protected from the unknown consequences of falling.

There are several advantages of group mounted systems. A group mounted system is less sensitive to variations in weights of individual items of equipment because of the larger combined weight of all items and the platform. With a number of items there is a greater flexibility of controlling the center of gravity of the total mass. In fact, ballast may be added to the platform to align the center of gravity with the principal axis to form a balanced system. A group mounted system generally requires less rattle space than several independently mounted items. Also, the interconnections between components is greatly simplified if they are all mounted on a single platform. Finally, an important advantage of group systems is cost. Individual mounts will require a large number of isolator units. Although larger, more costly, units are required for the group mounting system, fewer units are required and the cost per pound of supported load will be much lower.

6-47.3.2 *Platform Characteristics.* A platform for group mounted systems offers great flexibility in controlling the center of gravity of the supported masses to produce a balanced system where modes of vibration are uncoupled. Ballast may be securely anchored to the platform at locations which would move the center of gravity of the total mass to coincide with the elastic center of the isolation system. The determination of the weight and location of this ballast can be greatly simplified by uncoupling the effect of adding weight in the x and y directions of the principle elastic axes. This uncoupling can be accomplished by locating the ballast symmetrically about the x axis when moving the location of the center of gravity in the y direction. In this manner, the location of the center of gravity may be altered independently about the elastic center in the x and y directions. If for practical reasons the ballast cannot be located symmetrically about a principle axis, then the two directions must be considered simultaneously.

Providing additional ballast in excess of that requried to balance the platform provides for future changes in equipment or the addition of new equipment without actually changing the isolation system. The springs will not require replacement nor will the structural members of the platform need to be increased in size. Additional equipment is placed on the platform and ballast is removed and/or relocated to balance the new equipment arrangement. To provide for future eqipment changes, it is suggested that additional ballast equal to 25 percent of the weight of the equipment and the required ballast be distributed on the platform. The location of this ballast must not change the center of gravity of the existing balanced system. If future needs have been established, the platform and isolators would be designed for the future equipment. However, ballast would be provided to compensate for the weight of the future equipment and balance the system for the existing equipment.

The stiffness of the platform must be large enough to insure that the platform and associated group mounted equipment can be treated as a rigid body. This criterion is usually satisfied if the lowest natural frequency of any member of the platform is at least five (5) times the natural frequency of the spring mass system. When large, heavy items of equipment are involved, platforms meeting this stiffness criteria may not be practical. In such cases, the platform equipment configuration should be treated as a multi-mass system.

6-47.3.3 *Isolator Arrangements.* There are many ways to support a shock isolated item. Some desirable features have been discussed previously in connection with dynamically balanced systems. The isolators may be positioned in many ways. The more important factors affecting the selection of an isolator arrangement are:

1. The size, weight, shape and location of the center of gravity of the suspended mass;

2. The direction and magnitude of the input motions;

3. Rotation of the lines of action of the devices should be small over the full range of displacements of the system to avoid system nonlinearities;

4. Coupling of modes should be minimized;

5. Static and dynamic instability must be prevented;

6. It is desirable in most cases, and necessary in some, that the system return to its nominal position;

7. Space available for the isolation system; and

8. Type of isolation devices used.

Some of the more common isolator arrangements are shown in figures 6-57 and 6-58. The systems shown are assumed to have the same arrangements of isolators in a plane through the center of gravity (c.g.) and perpendicular to the surface of the page. The dynamically balanced system (intersection of the elastic axes and the principal inertia axes located at point A) shown in figure 6-56, is probably the least common of all suspension systems.

6-47.3.4 *Base-Mounted Isolation Systems.* In figure 6-57a, the mass is supported on four (4) isolators. These isolators must provide horizontal, vertical and rotational stiffnesses in order for the system to be stable under all possible motions. There will be coupling between horizontal displacements and rotations about horizontal axes. This arrangement and that shown in figure 6-57b are appropriate in those cases where there are no convenient supports for horizontal isolators.

The arrangement of figure 6-57b is preferred since the line of action of the isolators can be directed towards the c.g. of the mass to allow decoupling of some modes. As in the case of figure 6-57a, the isolators must possess adequate stiffness in axial and lateral directions to insure stability under static and dynamic conditions.

In figure 6-57c, the isolators are oriented parallel to the three orthogonal system axes. This arrangement provides system stability even when the isolators possess only axial stiffness. If the c.g. of the suspended mass is located as shown, decoupling of modes is possible. While the lines of action of the isolators pass through the c.g. under static conditions, response of the system to base motions will obviously alter its geometry. When the line of action of the isolators is changed due to displacement of the mass relative to its supports, coupling of the modes of vibration will be introduced. The degree of coupling is affected by the magnitude of the displacements and the length of the isolators. Consequently, isolator properties and arrangement should be selected so as to minimize the effects of displacements.

6-47.3.5 *Overhead Pendulum Shock Isolation Systems Using Platforms.* Two arrangements of overhead pendulum shock isolation devices using platforms to support the sensitive components are shown in figure 6-58. In both cases, the center of gravity of the suspended mass is relatively low. These types of suspension systems have been used extensively in protective structures for various conditions including individual small and large items, multiple items of various sizes as well as a combination of personnel and equipment supported on various sized platforms.

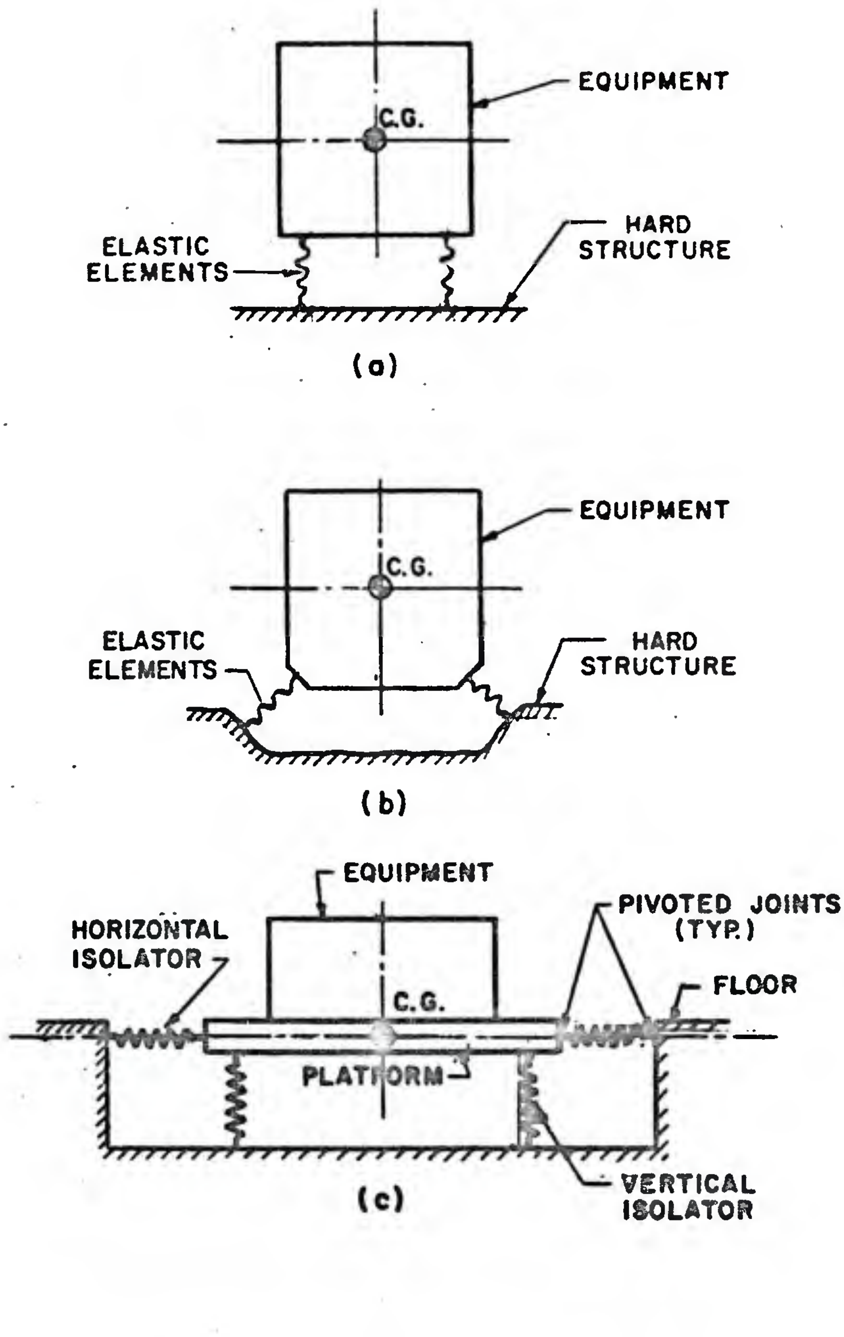


Figure 6-57 Base-mounted isolation systems configuraiton

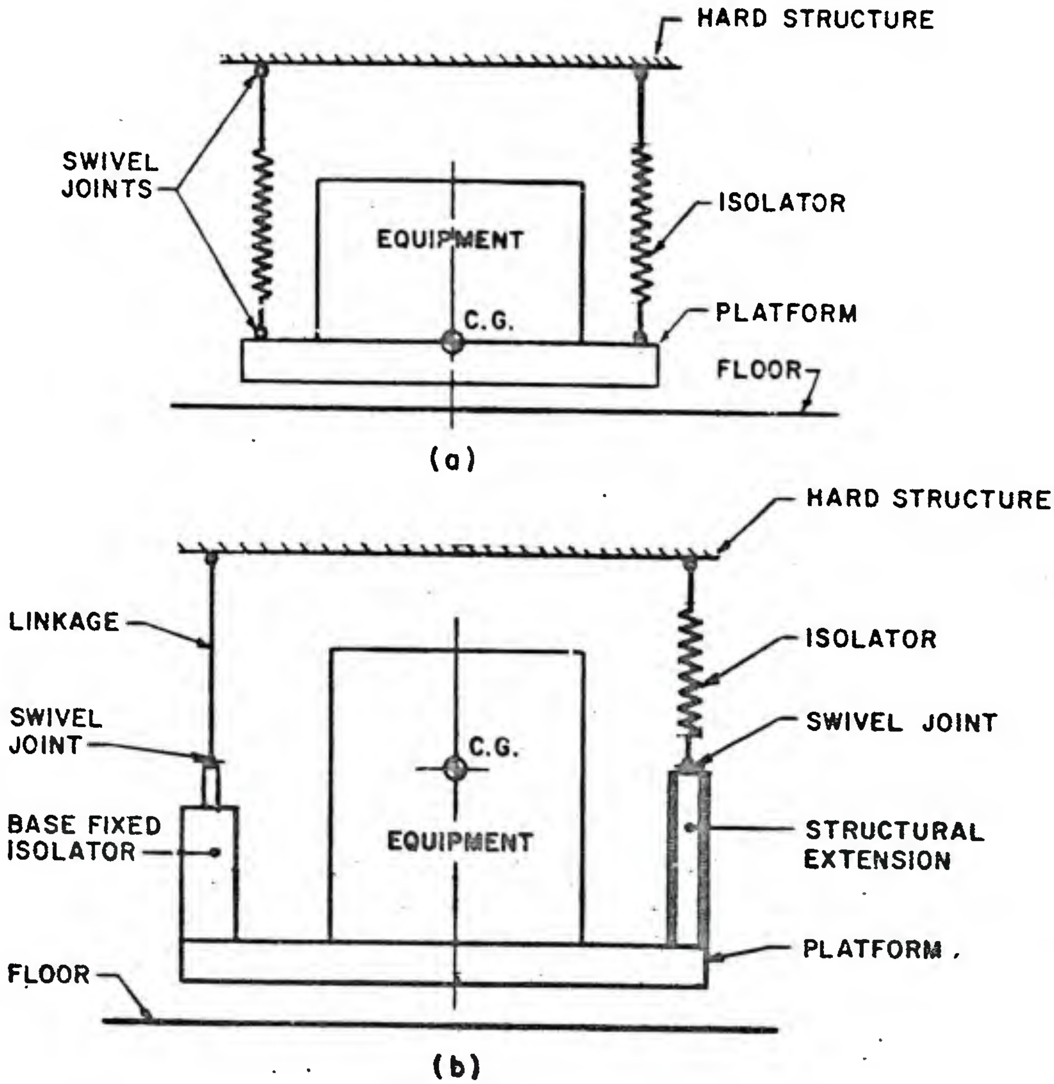


Figure 6-58 Overhead pendulum shock isolation systems using platforms

The overhead pendulum system normally uses swivel joints at the points of attachment so that the system may swing freely. Horizontal input motions cause the pendulum to swing. Gravity provides the horizontal restoring force or stiffness. This force is a function of the total weight of the suspended mass. The natural frequency of vibration of the pendulum is a function of the length of the pendulum and is given by:

$$f_n = \frac{1}{2\Pi} \left(\frac{386.4}{L}\right)^{1/2} \qquad 6\text{-}56$$

where

f_n = natural frequency of vibration

L = length of pendulum

Each pendulum arm includes an isolator which establishes the stiffness of the system in the vertical direction. These isolators can introduce nonlinearities and coupling between the pendulum and vertical spring modes. The system is linear for small angular displacements, that is, when the angular change θ of the pendulum arm from the vertical position is approximately equal to the sine of the angle ($\theta = \sin\theta$). The system can be considered uncoupled if the pendulum frequency is not near one half of the vertical spring frequency. If the pendulum frequency is in the vicinity of one half the vertical frequency, the interchange of energy between the modes can lead to pendulum motions greatly exceeding those predicted by linear assumptions.

In a shock spectra maximum displacements occur at low frequencies, maximum velocities at intermediate frequencies, and maximum accelerations at high frequencies. Since most pendulum systems have low natural frequencies, they are displacement sensitive. These systems attain maximum displacements and minimum accelerations. Consequently, they will normally require greater rattle space than other systems while at the same time providing maximum protection against horizontal accelerations at minimum costs. It should be realized that for H. E. explosions, maximum displacements are comparatively small and can be accommodated. One of the main advantages of overhead pendulum systems is that they do not require horizontal stiffness elements. Their attractiveness is greatly diminished in those cases requiring horizontal damping because of large motions.

The swivel joint attaching the pendulum arm to the platform determines the location of the horizontal elastic axis of the system. Figure 6-58b illustrates two ways of varying the point of attachment of the pendulum arm to the platform. The horizontal elastic axis is raised to coincide with the center of gravity of the suspended mass at the equilibrium position and help minimize coupling between modes of response. At the left side of the platform the isolator is contained in a housing rigidly attached to the platform. At the right side, a structural member is rigidly attached to the platform and the isolator is included in the pendulum arm.

In addition to supporting personnel and equipment, overhead pendulum systems can be used to shock-isolate building utilities. Individual utility runs may be isolated or several different utilities may be supported on a single platform. A single platform may cover an entire room and all building services may be supported. They would include a hung ceiling, lighting fixtures, utility piping, HVAC ducts, electrical cables and process piping. Of course, flexible connections must be used when connecting the services to the building or equipment.

6-48 Shock Isolation Devices

6-48.1 *Introduction*

A fundamental element of every shock isolation system is some sort of energy storage or energy dissipative device. These devices must be capable of supporting the items to be isolated under static and dynamic conditions and, at the same time, prevent transmission of any harmful shock loads to the items. In most cases, the isolator must have elastic force-displacement characteristics so that the system will return to a nominal equilibrium position after the dynamic loads have been applied. The desirable features of these devices include:

1. The dynamic force-displacement relationship of the isolator should be predictable for all directions in which it is required to provide stiffness.

2. The isolator should have low mass in order to minimize transmission of high frequency motions to the supported mass.

3. The frequency of the isolator should remain constant with changes in load, that is, its stiffness should vary in direct proportion to the load it supports. This allows the system to remain dynamically balanced throughout changes in the position of the supported mass.

4. The static position of the isolator should be adjustable so that the system can be leveled and returned to its nominal position should the suspended load change.

5. The isolator should have high reliability, long service life and low cost.

The various types of isolators used in most protective construction applications possess these characteristics in varying degrees. Any real isolator has some mass, and in some applications, the mass can be quite large and must be considered in the final analysis. Nonlinear force-displacement characteristics are often accepted to gain some other advantage. In energy dissipative systems, it may be necessary to provide other means of restoring the system to its original position. In general, most devices are some compromise combination of the desirable features which best suit the particular design situation.

The inclusion of energy dissipative (damping) devices in the isolation system offers several significant advantages, that is, damping can:

1. Reduce the severity of output motion response;

2. Reduce the effect of coupling between modes, thus reducing rattle space requirements;

3. Restore the system to an equilibrium position more quickly;

4. Decrease the sensitivity of the system to variations in input motions.

Damping can be provided internally in some isolation devices such as in liquid springs, but must be added externally in others such as those systems using helical coil springs. Different types of damping offer advantages and disadvantages which must be evaluated in the design process. A damping device may be effective in attenuating low frequency components of input motions but can increase the severity of high frequency components. Also, a damping device could prevent the system from returning to its nominal equilibrium position. Thus, care must be exercised in either designing a system employing isolator devices possessing inherent damping characteristics or adding damping devices, if the isolation system is to perform properly.

There are numerous types of isolators which can be used to accomplish the shock isolating function. In the design of protective structures for H. E. explosions, the induced building motions are not usually severe and the maximum building displacements are relatively small. As a result, shock isolation systems using helical coil springs (fig. 6-59) are by far the most common system employed. The reasons for the extensive use of helical springs should be obvious from the discussion below. Other shock isolation devices which may also be used are presented, in less detail, below.

It should be noted that the protective design engineer does not furnish the design for the shock isolator. The engineer designs the shock isolation system to be used but does not design the isolators (in most cases, a helical coil spring). Rather, specifications are furnished which define the desired characteristics of the isolator. For a helical spring, the specifications may include some or all of the following: maximum load, maximum static deflection, maximum dynamic deflection, spring stiffness, maximum height, maximum diameter, and factors of safety regarding allowable stresses and bottoming of the spring. It must be realized that as the number of specified parameters increase, the options available to the spring manufacturer are decreased.

6-48.2 *Helical Coil Springs*

A helical coil spring is fabricated from bar stock or wire which is coiled into a helical form. Figure 6-59 illustrates several spring mounts.

The helical coil spring has numerous advantages and comparatively few disadvantages. The advantages are that the spring is not strain-rate sensitive, self-restoring after an applied load has been removed, resists both axial and lateral loads, linear spring rate and requires little or no maintenance. For most applications, the coil spring usually requires a larger

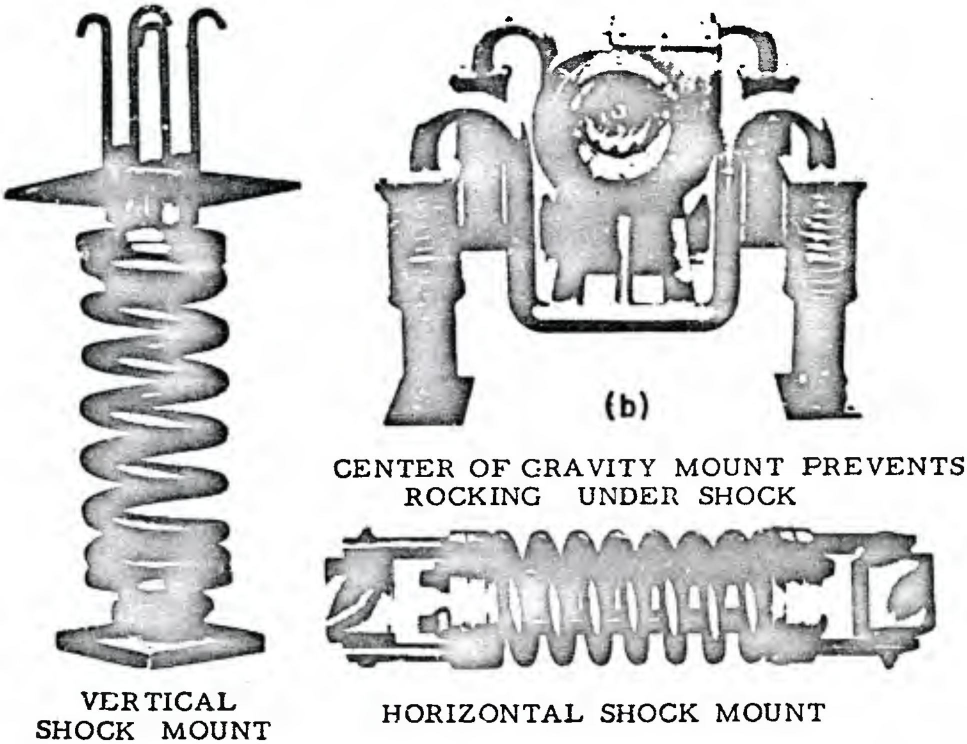


Figure 6-59 Helical compression spring mounts

space compared to other available shock isolators, and the spring cannot be adjusted to compensate for changes in loading conditions. If the weight of the supported object is changed, it is necessary to either change the spring or add additional springs. For most purposes, the helical coil spring can be considered to have zero damping. If damping is required, it must be provided by external means.

Helical coil springs may be used in either compression or extension. The extension springs are not subject to buckling and may offer a more convenient attachment arrangement. However, extension spring attachments are usually more costly and cause large stress concentrations at the point of attachment. For shock isolation applications, coil springs are generally used in compression. Buckling which can be a problem with compression springs, can be overcome by proper design or through the use of guides which are added either internally or externally to the coils. The discussion below will be concerned primarily with compression springs unless otherwise stated.

Helical coil springs may be mounted in two ways, the ends are either clamped or hinged. In most shock isolation applications, the spring ends are clamped since this method greatly increases the force required to buckle the spring. If space is at a premium, the energy storage capacity may be increased by nesting the springs (placing one or more springs inside the outermost spring). When nesting springs, it is advisable to alternate the direction of coils to prevent the springs from becoming entangled.
Although permanent set may be acceptable in some instances, it is normally required that the system return to its original position after being loaded. This can be accomplished in various ways, but the most common approach in the case of helical coil springs is to prevent inelastic action of the spring.

Helical coil springs are capable of resisting lateral load. While it is possible to use springs in this application, care should be exercised. There are possible arrangements which avoid subjecting the springs to this type of loading.

While the actual design of the helical coil spring is done by the manufacturer, the engineer must be certain that the springs he is specifying can actually be obtained and the space he has allocated for the springs are sufficient. Therefore, preliminary spring sizes must be obtained by the engineer to suit his intended application. It is suggested that available manufacturer's data be used for this purpose.

6-48.3 *Torsion Springs*

Torsion springs provide resistance to torque applied to the spring. In shock isolation applications, the torque is usually the result of a load applied to a torsion lever which is part of the torsion spring system. A typical torsion spring shock isolation system is illustrated in figure 6-60.

Since the axis of a torsion spring is normal to the direction of displacement, it can be used advantageously when space in the direction of displacement is limited. Torsion springs have linear spring rates, are not strain-rate sensitive, are self-restoring, and require little or no maintenance.

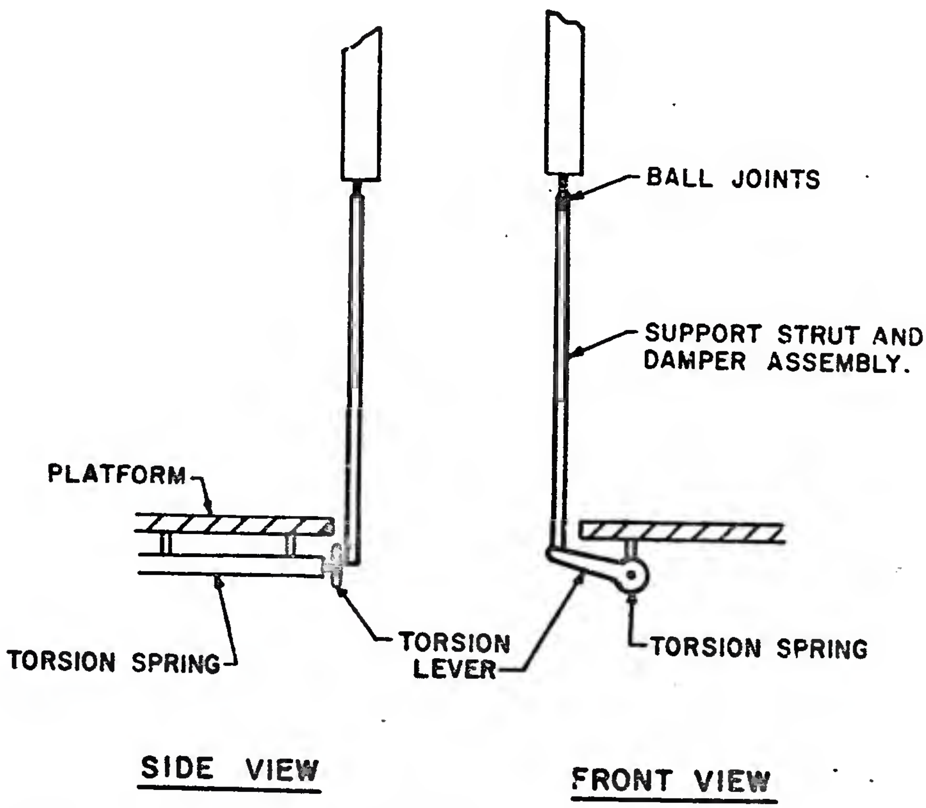


Figure 6-60 Typical torsion spring shock isolation system

Torsion springs can not be adjusted to compensate for changes in weight of shock isolation equipment, and damping must be provided by external means. The axial length of some types may preclude their use when space is limited.

There are three basic types of torsion springs; (1) torsion bars, (2) helical torsion springs, and (3) flat torsion springs. The type to be used will depend upon the space available and the capacity required. The torsion bar is normally used for light to heavy loads, the helical torsion spring for light to moderate loads, and the flat torsion spring for light loads. The torsion bar is the type most commonly found in protective structure applications and is most commonly used where large loads must be supported.

6-48.4 *Pneumatic Springs*

Pneumatic springs are springs whose action is due to the resiliency of compressed air. They are used in a manner similar to coil springs. The two basic types are the pneumatic cylinder with single or compound air chambers and the pneumatic bellows. The pneumatic cylinder is shown schematically in figure 6-61.

Pneumatic springs have the advantage of being adjustable to compensate for load changes. The spring rate can be made approximately linear over one range of deflection but will be highly nonlinear over another. They are quite versatile due to the variety of system characteristics which can be obtained by regulation of the air flow between the cylinder chamber and the reservoir tank. Some of the possible variations include:

1. Velocity-sensitive damping by a variable orifice between chamber and reservoir;

2. Displacement-sensitive damping by a variable orifice controlled by differential pressure between chamber and reservoir;

3. A nearly constant height maintained under slowly changing static load by increasing or decreasing the system air content using an external air supply and a displacement-sensitive servo-system controlling inlet and exhaust valves;

4. A constant height under widely varying temperatures achieved by the same system described for maintaining a constant height.

The disadvantages of pneumatic springs include higher cost and more fragile construction. They have a limited life span in comparison to mechanical springs and must be maintained. Also these springs provide resistance for axial loads only.

6-48.5 *Liquid Springs*

A liquid spring consists of a cylinder, piston rod, and a high pressure seal around the piston rod. The cylinder is completely filled with a liquid, and as the piston is pushed into the cylinder, it compresses the liquid to very high pressures.

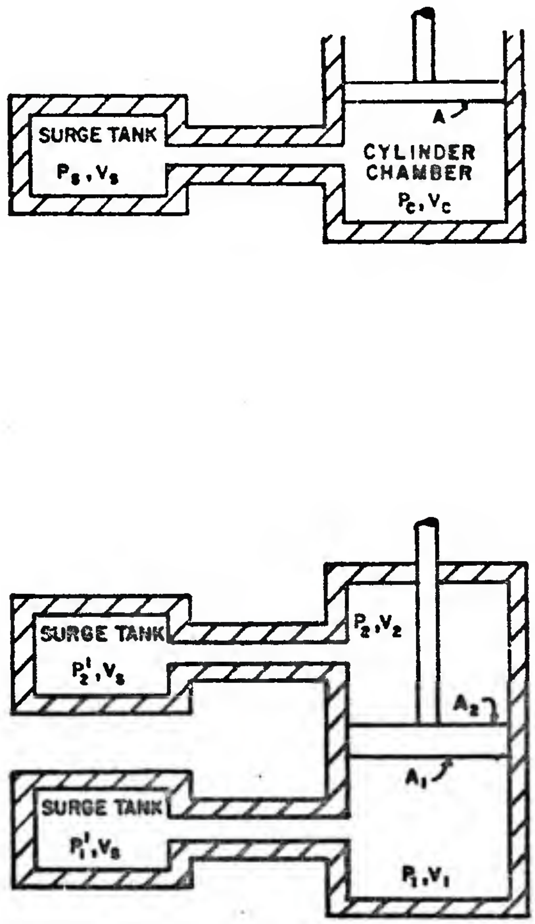


Figure 6-61 Schematic of single and double acting pneumatic cylinders

The configurations of liquid springs are divided into three major classes according to the method of loading. The classes are simple compression, simple tension and compound compression-tension. Although they are loaded in different ways, all three types function as a result of compression of the liquid in the cylinders. Schematics of the tension and compression types are shown in figure 6-62. The compound spring is merely a more complex mechanical combination of the two basic types. The tension type is the more common in protective construction applications. The cylinders are often fitted with ported heads to guide the piston and provide damping. Damping can also be provided through the addition of drag plates to the piston rods.

Liquid springs are very compact devices with high, nearly linear, spring rates. They can be adjusted to compensate for load changes, are self-restoring and can absorb larger amounts of energy. They are highly sensitive to changes in temperature and fluid volume changes. Because liquid springs normally operate at high pressures, high quality, close tolerance seals are required around the piston. Friction between the seal and piston provides appreciable damping and increases the spring rate from 2 to 5 percent. Liquid springs are high pressure vessels requiring high quality materials and precision machine work, and as a result, they are expensive. However, they are difficult to equal as compact energy absorption devices.

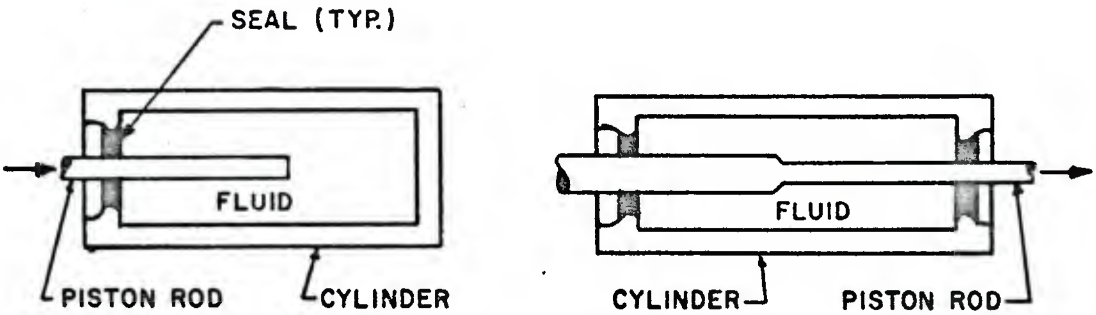
6-48.6 *Other Devices*

6-48.6.1 *Introduction.* The helical, torsion, pneumatic and liquid springs are the more common types of isolators for larger masses. There are other devices especially suited for particular applications and smaller loads. Some of these isolators are discussed below.

6-48.6.2 *Belleville Springs.* Belleville springs, also called Belleville washers or coned-disc springs, are essentially spring steel washers which have been formed into a slightly conical shape. A typical Belleville spring is illustrated in figure 6-63.

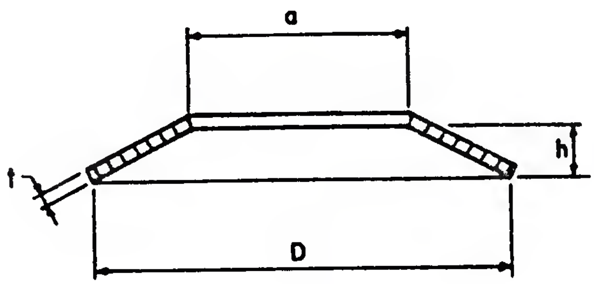
The main advantage of Belleville springs over other types of springs is the ability to support large loads at small deflections with mimimum space requirements in the direction of loading. They are useful in applications requiring limited shock attenuation and as back up systems to reduce shock in the event of bottoming of coil springs. They are relatively inexpensive and readily available in capacities up to 60,000 pounds. Changes in loading conditions are accommodated by the addition or removal of units.

6-48.6.3 *Flat Springs.* A flat spring is simply a steel beam or plate whose physical dimensions and support conditions are varied to provide the desired force displacement relationship. The two basic configurations are the simple spring with one element and leaf springs with multiple elements. Flat springs normally require only a limited amount of space in the direction of displacement and provide linear, non-strain-rate sensitive and self-restoring spring. They require little or no maintenance. Single element flat springs can be considered to have no damping while leaf springs will exhibit some damping due to the friction between individual elements.



(a) COMPRESSION (b) TENSION

Figure 6-62 Schematic of liquid springs



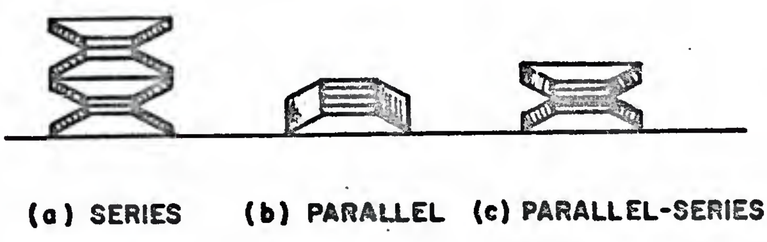


Figure 6-63 Belleville springs

6-48.6.4 *Solid Elastomer Springs.* Solid elastomer springs are made from rubberlike materials. They are often called shock mounts because of their wide use in shock isolation applications. They are normally used in medium to light duty applications and represent an economical solution to the isolation of small items of equipment. However, these springs will allow only small displacements. These springs are fabricated from a wide variety of natural and synthetic rubbers and compounds and in numerous sizes and shapes to satisfy a wide range of applications. Because of the range in capacity and characteristics of commercially available units, only in unusual cases is it necessary to design a unit.

In most applications, the solid elastomer spring will require little space and exhibits good weight to energy storage ratios. Use of these springs requires consideration of the operating environment. The desirable properties of some elastomers can be significantly degraded when exposed to low or high temperatures, sunlight, ozone, water or petroleum products.

The response of elastomeric springs is nonlinear in most applications because of the nonlinear stress-strain properties of elastomers. The springs are self-damping because of the viscoelastic properties of the elastomers. They are almost always in compression because of bonding limitations. These springs will only permit comparatively small displacements.

6-49 Hardmounted Systems

Some items of equipment do not require shock isolation because the predicted motions at their point of attachment to the supporting structure does not exceed their shock tolerance. Those items can normally be hardmounted to the supporting structure. A hardmount is a method of attachment which has not been specifically designed to provide a significant reduction in the input motions to the equipment. Since all methods of attachment exhibit some flexibility, there is no precise division between shock isolators and hardmounts. Both types of devices will modify input motions to some degree. However, the modification of input motions produced by hardmounts will generally be small while shock isolators can greatly affect these motions.

In contrast to shock isolation systems, hardmounted systems will normally exhibit natural frequencies much higher than those corresponding to the lower modes of vibration of the supporting structure. Although this characteristic offers the advantage of reduced rattlespace, it also provides for the more efficient transmission of higher frequency components of the suport structure motion to the attached item. Thus, it would appear that a more exact structural analysis is required for hardmounted systems in order to include higher modes of vibration. In practice, the need for exact analyses is at least partially offset by higher factors of safety in mount design and equipment shock tolerance. However, such an approach can lead to unrealistic attachment designs. A more practical approach is to choose, or design, attachments which limits the fundamental frequency of the hardmounted system. A lower frequency system provides some attenuation of higher frequency input motions, and reduces the possibility of resonance with high frequency motions resulting from stress wave reflections within structural elements. Although the choice of a natural frequency will depend on the properties of the supporting structure and the hardmounted equipment, fundamental frequencies in the range of 10 to 1000 cycles per second are reasonable for most applications.

The approach chosen for hardmount design is normally a combination of higher safety factors and the use of lower frequency systems. The design will be based upon considerations of cost, importance of the item supported, the size and weight of the item, and the consequence of failure of the attachment system.

The use of shock spectra to define the input motions of hardmount systems is considered adequate for final design of all simple hardmount systems of a non-critical nature. It is also considered adequate for preliminary design of critical systems and those whose representation as a single degree of freedom system is questionable. However, it is recommended that the final design be performed using a more exact dynamic analysis wherever practical.

6-50 Attachments

6-50.1 *Introduction*

In a shelter type structure subjected to air blast and ground shock effects, all interior contents must be firmly attached to the structure. This attachment insures that the building contents will not be dislodged and become a source of injury to personnel or damage to critical equipment. The building contents would include not ony equipment which is either shock isolated or hardmoutned (attached directly to structure) but also the building utilities as well as interior partitions and hung ceilings. The building utilities would include all piping (such as process, potable water, sanitary, fire protection, etc.), HVAC ducts, electrical cables, light fixtures and electrical receptacles.

6-50.2 *Design Loads*

An object subjected to a shock loading produces an inertial force which acts through its center of gravity. The magnitude of this force is given by:

$$F = Wa$$

where

F = inertial force

W = Weight of object

a = acceleration in g's

Accelerations may be imparted to the object in one or more directions producing inertial forces in the respective directions. These inertial forces are resisted by the reactions developed at the object's supports. All inertial forces are assumed to be acting on the object concurrently. The support reactions are obtained by considering the static equilibrium of the system.

APPENDIX 6A ILLUSTRATIVE EXAMPLES

Problem 6A-1 Masonry Wall Design

Problem: Design a reinforced masonry wall for an exterior blast load.

Procedure:

Step 1. Establish design parameters:

a. Pressure-time loading.

b. Structural configuration including geometry, support conditions and type of wall (i.e. reusable or non-reusable).

Step 2. Select masonry unit size and the size and type of reinforcement. Assume the distance between the tension and compression reinforcement (d_c). Also determine the static design stresses for the masonry unit and the reinforcement (Section 6-8.1). The average yield stress of reinforcement is increased 10 percent.

Step 3. Calculate the dynamic design stress of the reinforcement, using the static stresses from Step 2 and the dynamic increase factors from Section 6-8.2 (For joint reinforced masonry construction the compressive strength of the concrete may be ignored. See Section 6-8.3).

Step 4. For the size and type of reinforcement selected in Step 2, calculate the area of reinforcement per unit width of the wall. Using the value of d_c from Step 2, the dynamic design strength from Step 3 and the area of reinforcement from above, determine the ultimate moment capacity of the wall (eq. 6-2).

Step 5. Determine the ultimate resistance of the wall using the ultimate moment capacity of Step 4 and the equations of table 3-1 (if the wall is a one-way spanning element) or table 3-2 or 3-3 (if the wall spans two directions).

Step 6. From table 6-3, find the moment of inertia of the net section I_n. Calculate the moment of inertia of the cracked section I_c, using equation 6-7 and the value of d_c from Step 2. Determine the average moment of inertia, using the values of I_g and I_c from above and equation 6-6.

Step 7. Calculate the modulus of elasticity of the masonry unit E_m from equation 6-1 and the masonry unit strength of Step 2.

Step 8. If the wall spans one direction only, use the average moment of inertia from Step 6, the modulus of elasticity from Step 7 and the equations from table 3-8, to find the equivalent elastic stiffness. For a two-way spanning wall, use the methods of Section 3-13 to calculate the equivalent elastic stiffness.

Step 9. Determine the equivalent elastic deflection using the ultimate resistance (Step 5), the equivalent stiffness (Step 8) and equation 3-36.

Step 10. Find the load-mass factor K_{LM}, for the elastic, elasto-plastic and plastic ranges from table 3-12 or 3-13. Average the values of K_{LM} for the elastic and elasto-plastic ranges. Average that value with the K_{LM} for the plastic range to find the value of K_{LM} to be used for the elment. Calculate the unit mass of the masonry unit and multiply the unit mass by K_{LM} to obtain the effective unit mass of the wall.

Step 11. Calculate the natural period of vibration from equation 3-60, the effective mass from Step 10 and the equivalent stiffness from Step 8.

Step 12. Determine response chart parameters:

a. Peak dynamic loading P (Step 1).
b. Ulitmate resistance r_u (Step 5).
c. Duration of load T (Step 1).
d. Natural period of vibration T_N (Step 11).

Calculate the ratio of ultimate resistance to peak dynamic loading (r_u/P) and duration of load to natural period (T/T_N). Using these ratios and the appropriate figures (figs. 3-54 through 3-266) determine the ductility ratio X_m/X_E.

Step 13. Compute the maximum deflection X_m by multiplying the ductility ratio by the elastic deflection of Step 9. From table 3-5 (for a one-way spanning wall. For a two-way spanning wall use table 3-6) and the value of X_m, find the maximum support rotation. Find the maximum rotation permitted from table 6-2 and compare with the rotation calculated above. If rotation is larger than that permitted, repeat steps 1 through 13.

Step 14. Determine the ultimate shear stress at $d_c/2$ from the support. Using equation 6-4 or 6-5, compute the area of shear reinforcement required for the above shear stress.

Step 15. Using X_m/X_E and T/T_N (both values from step 12) find the rebound resistance from figure 3-268. With rebound resistance and the equations of table 3-9, 3-10 or 3-11, calculate the rebound shear. Then compute the area of anchor reinforcing required using the rebound shear from above and the dynamic strength of the reinforcement from step 3.

Example 6A-1 Masonry Wall Design

Required: Design a joint reinforced masonry wall supported by steel columns for an exterior blast load.

Solution:

Step 1. Given:

a. Pressure-time loading (fig. 6A-1).

b. Wall spans in one direction only, is rigidly supported at both ends, and has a clear span between columns of 150 inches. The wall is part of a reusable structure.

Step 2. Use 12" wide hollow concrete masonry units and ladder type reinforcing with No. 8 Gage side rods and No. 9 Gage cross rods 16" o.c. Assume d_c = 10" for this type of reinforcing.

For hollow concrete masonry units the static compressive stress (f'_m) is 1350 psi. For joint reinforcement the yield stress of 70,000 psi is increased 10 percent to 77,000 psi.

Step 3. Calculate dynamic design stresses of the reinforcement:

a. Dynamic Increase factors

- flexure: 1.17
- shear: 1.00

b. Dynamic strengths

- flexure f_{dy} = 1.17 X 77,000 = 90,090 psi

- shear f_{dy} = 1.0 X 77,000 = 77,000 psi

Step 4. Determine the ultimate moment capacity of the wall.

a. Calculate the area of reinforcement per unit width of the wall.

Use one layer of reinforcement between every masonry unit joint, therefore 8 inches o.c.

$A_s = 0.0206/8 = 0.0026 \text{ in}^2/\text{in}$

b. Ultimate moment capacity (eq. 6-2).

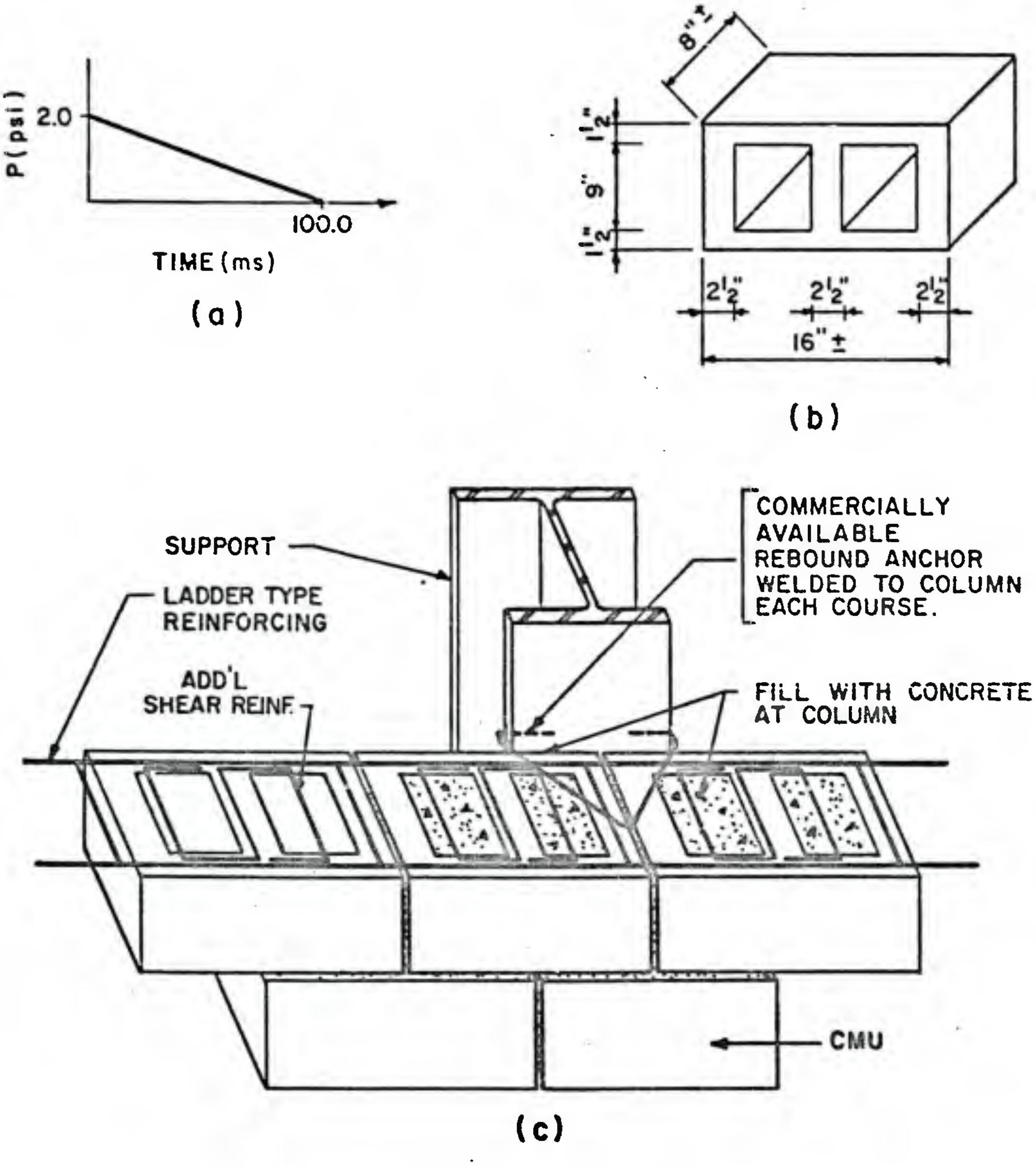
$$M_u = A_s f_{dy} d_c$$

$$= 0.0026 \times 90{,}090 \times 10$$

$$= 2342 \text{ in-lbs/in}$$

Step 5. Calculate ultimate resistance (table 3-1)

$$r_u = \frac{8(M_N + M_P)}{L^2}$$



P (psi)
2.0
100.0
TIME (ms)
(a)
8"±
1½"
9"
1½"
2½"
2½"
2½"
16"±
(b)
SUPPORT
LADDER TYPE REINFORCING
ADD'L SHEAR REINF.
COMMERCIALLY AVAILABLE REBOUND ANCHOR WELDED TO COLUMN EACH COURSE.
FILL WITH CONCRETE AT COLUMN
CMU
(c)

Fig. 6A-1

$$= \frac{8(2342 + 2342)}{150^2}$$

$$= 1.66 \text{ psi}$$

Step 6. Find the average moment of inertia.

a. Moment of inertia of net section (table 6-3).

For a 12 inch unit

$$I_n = 83.3 \text{ in}^4/\text{in}$$

b. Moment of inertia of cracked section (eq. 6-7).

$$I_c = 0.005 \times d_c^3$$

$$= 0.005 \times 10^3$$

$$= 5.0 \text{ in}^4/\text{in}$$

c. Average moment of inertia (eq. 6-6).

$$I_a = \frac{I_n + I_c}{2}$$

$$= \frac{83.3 + 5.0}{2}$$

$$= 44.2 \text{ in}^4/\text{in}$$

Step 7. Compute modulus of elasticity of the masonry unit (eq. 6-1).

$$E_m = 1000 f'_m$$

$$= 1000 \times 1350$$

$$= 1.35 \times 10^6 \text{ psi}$$

Step 8. Determine the equivalent elastic stiffness (table 3-8).

$$K_E = \frac{307 E_m I_a}{L^4}$$

$$= \frac{307 \times 1.35 \times 10^6 \times 44.2}{150^4}$$

$$= 36.19 \text{ psi/in}$$

Step 9. Calculate the equivalent elastic deflection (eq. 3-36).

$$X_E = r_u/K_E$$

$$= 1.66/36.19$$

$$= 0.046 \text{ in}$$

Step 10. Calculate the effective unit mass of the wall.

a. Find the average load-mass factor K_{LM} (table 3-12)

Elastic $K_{LM} = 0.77$
Elasto-plastic $K_{LM} = 0.78$
Plastic $K_{LM} = 0.66$

For limited plastic deflections

$$K_{LM} = [(0.77 + 0.78)/2 + 0.66]/2 = 0.72$$

b. Determine the unit mass of wall.

Using table 6-1

$$W = \frac{[(16 \text{ X } 12) - 2(4.25 \text{ X } 9)]}{16} \text{ X } \frac{150 \text{ pcf}}{12^3} = 0.627 \text{ psi}$$

$$m = \frac{w}{g} = \frac{0.627}{32.2 \text{ X } 12 \text{ X } 10^{-6}} = 1622.7 \frac{\text{psi-ms}^2}{\text{in}}$$

c. Calculate effective unit mass.

$$m_e = K_{LM}m$$

$$= 0.72 \text{ X } 1622.7$$

$$= 1168.3 \text{ psi-ms}^2/\text{in}$$

Step 11. Determine the natural period of vibration (eq. 3-60).

$$T_N = 2\Pi(m_e/K_E)^{1/2}$$

$$= 2\Pi(1168.3/36.19)^{1/2}$$
$$= 35.7 \text{ ms}$$

Step 12 Determine the response of the wall.

a. Calculate design chart parameters.

$$T/T_N = 100.0/35.7 = 2.80$$

$$r_u/P = 1.66/2.0 = 0.83$$

b. From figure 3-54

$\mu = X_m/X_E = 8.0$

Step 13. Check support rotation.

a. Compute maximum deflection.

$X_m = \mu X_E$

$= 8.0 \times 0.046$

$= 0.368$ in

b. Calculate support rotation (table 3-5)

$\theta = \tan^{-1}(2X_m/L)$

$= \tan^{-1}(2 \times 0.368/150)$

$= 0.28°$

c. Compare rotation with criteria.

From table 6-2

$\theta = 0.5° > 0.28°$ O.K.

Step 14. Design shear reinforcement.

a. Calculate shear force $d_c/2$ from support.

$$V_u = \frac{ru(L-d_c)}{2}$$

$$= \frac{1.66(150-10)}{2}$$

$= 116.2$ lb/in

b. Find net area of section from table 6-1.

$A_n = 2b \times$ Face thickness/b

$= 2 \times 8 \times 1.5/8$

$= 3.0\ in^2/in$

c. Compute ultimate shear stress (eq. 6-3)

$$v_u = \frac{V_u}{A_n}$$

$$= \frac{116.2}{3.0}$$

$$= 38.73 \text{ psi}$$

d. Find area of shear reinforcement required (eq. 6-4)

Assume s = 4".

$$A_v = \frac{v_u bs}{\Phi f y}$$

$$= \frac{38.73 \times 8 \times 4}{0.85 \times 77{,}000}$$

$$= 0.0189 \text{ in}^2$$ Use No. 8 Gage

Use 3 legs of No. 8 gage wire at 4" o.c. between each cross rod.

Step 15. Design rebound anchor ties.

a. Find rebound resistance force.

$T/T_N = 2.8$

$X_m/X_E = 8.0$

From figure 3-268

$r^-/r_u = 0.47$

$r^- = 0.47 \times 1.66$

$= 0.78$ psi

b. Calculate rebound shear at support

$$V_r = \frac{r^- L}{2}$$

$$= \frac{0.78 \times 150}{2}$$

$$= 58.5 \text{ lb/in}$$

c. Required area of anchor reinforcing

$$A_r = \frac{V_r}{f_{dy}}$$

$$= \frac{58.5}{90{,}090} = 0.00065 \text{ in}^2/\text{in}$$

Use one anchor between every masonry unit joint, therefore 8" o.c.

$$A_r = 0.00065 \times 8$$

$$= 0.0052 \text{ in}^2/\text{anchor}$$

Use 3/16" diameter triangular ties.

Problem 6A-2 Design of Prestressed Precast Element

Problem: Design a prestressed precast element for a given blast load.

Procedure:

Step 1. Establish design parameters:

a. Pressure-time loading.

b. Material strength.

c. Span length.

d. Static loads.

e. Deflection criteria.

Step 2. Select a standard precast section from the PCI Design Handbook or manufacturer's catalogs. Also select a standard strand pattern.

Step 3. Determine dynamic increase factors for concrete and reinforcement from paragraph 6-12. Increase the static design stress of the reinforcing bars and welded wire fabric 10 percent for the average yield stress. Calculate the dynamic design design stresses using the above DIF and the static stresses.

Step 4. From Volume IV and a unit weight of concrete equal to 150 pcf, calculate the modulus of elasticity for concrete. With the above modulus for concrete and those for reinforcing bars and prestressing tendons, calculate the modular ratio.

NOTE: If the section has a flange (e.g. a single or double tee section) it must be designed according to the principles of Volume IV. The flange is a one-way, non-prestressed slab spanning between webs. The critical section is usually a cantilever.

Step 5. Calculate the properties of the section:

a. Gross area.

b. Gross moment of inertia.

c. Unit weight.

d. Distance from extreme compression fiber to the centroid of the prestressing tendon, d_p.

e. Area of prestressing tendons.

f. Prestressed reinforcement ratio (eq. 6-23).

Step 6. Given the type of prestressing tendon, determine γ_p. Using the dynamic concrete stress from step 3, calculate β_1. Using equation 6-27, 6-28 or 6-29 and the values of γ_p and β_1 from above, calculate the average stress in the prestressing tendon.

Step 7. With the reinforcement ratio from step 5e, the average stress in the prestressing tendon and the value of β_1 from step 6, and the dynamic strength of materials from step 3, check that reinforcement ratios are less than the maximum permitted by equation 6-30 or 6-31.

Step 8. Using the area of reinforcement and the value of d_p from step 5, the dynamic stresses of step 3, and the average stress in the prestressing tendon from step 6, calculate the moment capacity of the element (eqs. 6-20 and 6-21).

Step 9. With the equations of table 3-1 and the moment capacity of step 8, calculate the ultimate unit resistance. As precast buildings are only subject to low blast pressures, the static loads become significant. To determine the resistance available to resist the blast load, subtract the static dead and live loads from the ultimate unit resistance.

Step 10. Calculate the moment of inertia of the cracked section I_c, using equation 6-33, the modular ratio from step 4 and the area of prestressing tendons, the value of d_p and the prestressed reinforcement ratio from step 5. Using this value of I_c and the gross moment of inertia of step 5b, find the average moment of inertia from equation 6-32.

Step 11. Using the equations of table 3-8, the modulus of elasticity for concrete from step 4 and the average moment of inertia from step 10, calculate the elastic stiffness of the section.

Step 12. Determine the load-mass factor K_{LM}, in the elastic range for the appropriate loading condition from table 3-12. Also, calculate the unit mass of the section and multiply it by K_{LM} to obtain the effective unit mass of the element.

Step 13. With equation 3-60, the effective mass from step 12 and the elastic stiffness from step 11, calculate the natural period of vibration T_N, of the section.

Step 14. Determine the response chart parameters:

a. Duration of load T (step 1).

b. Natural period of vibration T_N (step 13).

Calculate the ratio T/T_N and using this ratio, determine the dynamic load factor DLF from figures 3-49 through 3-53. Section must remain elastic, and hence the actual resistance obtained by the element (which is equal to the peak dynamic load from step 1 multiplied by the DLF) must be less than the resistance available. If the section does not remain elastic steps 1 through 14 must be repeated.

NOTE: For bilinear loads, calculate the ratio of the peak dynamic load P to the resistance available. Using P/r_u and the value of T/T_N calculated above, enter the appropriate response chart (figs. 3-64 through 3-266) and find the ductility ratio X_m/X_E. For the section to remain elastic the ductility ratio must be less than or equal to one.

Step 15. Calculate the deflection of the element X_m by adding the dead and live loads (step 1), and the resistance obtained by the structure (step 14) and dividing by the elastic stiffness (step 11). Using the deflection and the equations of table 3-5, determine the support rotation. Compare the rotation with deflection criteria of step 1e. If comparison is satisfactory continue with step 16. If comparison is not satisfactory repeat steps 1 through 15.

Step 16. Calculate the elastic deflection X_E from equation 3-36, using the ultimate resistance of step 9 and the elastic stiffness of step 11. Then calculate the ductility ratio X_m/X_E using the value of X_m from step 15. With the ductility ratio and the ratio T/T_N from step 14, enter figure 3-268 and find the percentage of rebound. Extrapolate if necessary.

Step 17. Find the required rebound resistance by multiplying the ultimate unit resistance by the ratio from step 16 and subtracting the dead load. In no case should the required rebound resistance be less than half the resistance available during the loading phase. With equations of table 3-1 and the required rebound resistance, find the required rebound moment capacity.

Step 18. Calculate an approximate value of d^{-} and the amount of concrete strength available for rebound (eq. 6-35). Assume a depth of the equivalent rectangular stress block and, by a trial and error method, using equation 6-34 find the area of rebound reinforcement required. Check that the amount of reinforcement does not exceed the maximum permitted by equation 6-36 or 6-37.

Step 19. Using the equations of table 4- , the value of the d_p of step 5c and the total load (which is the resistance obtained by the structure plus the dead and live loads) calculate the shear stress at d_p away from the support. Also calculate the allowable shear stress on an unreinforced web using the dynamic design stress (step 16), and equation 4- . Design the shear reinforcement according to the provision of Section 4- .

Step 20. Calculate the shear at the support V_d from the equations of table 3-9 and the value of the total load. Calculate the maximum allowable direct shear using the dynamic concrete strength and equation 4- . Compare the allowable shear with V_d. If V_d is greater than the size of the section must be increased.

Step 21. Check if section is adequate for service loads using the PCI Design Handbook and the ACI Code.

Example 6A-2 Design of a Prestressed Precast Roof Panel

Required: Design a prestressed precast roof subject to an overhead blast load. Use a double tee section.

Solution:

Step 1. Given:

a. Presure-time loading (fig 6A-2).

b. Material strenghts

Concrete: $f'_c = 5000$ psi

Prestressing Steel: $f_{pu} = 270{,}000$ psi
$f_{py}/f_{pu} = 0.85$

Welded Wire Fabric $f_y = 65{,}000$ psi

Reinforcing Bars $f_y = 60{,}000$ psi

c. Span length is 40 ft. = 480 in.

d. Live load is 15 psf

e. Maximum ductility ratio ≤ 1.0

Maximum support rotation $\leq 2°$

Step 2. Select a double tee section and strand pattern.

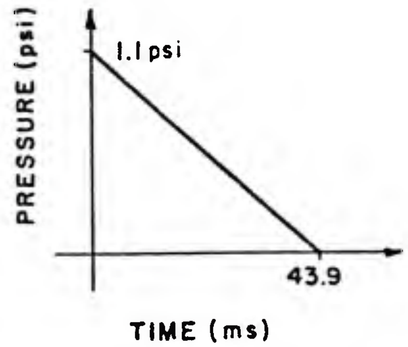
Try 8DT24 (fig. 6A-2)

Section properties from PCI Design Handbook:

$A = 401$ in^2 $\quad$ $y_t = 6.85$ in

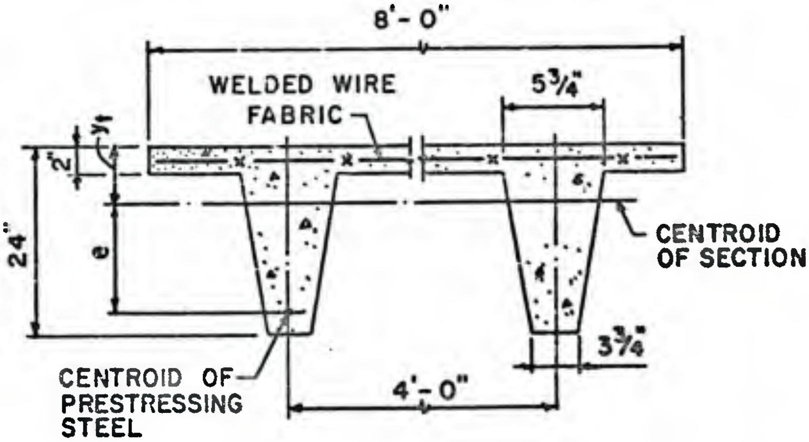
$I_g = 20{,}985$ in^2 $\quad$ $w = 418$ lb/ft.

Try strand pattern 48-S, two 1/2 inch diameter straight strands in each tee.



(a)

PRESSURE – TIME LOADING



(b)

DOUBLE TEE SECTION

Fig. 6A-2

Area of each strand = 0.153 in^2

e = 14.15 in

Step 3. Determine design stresses.

a. Dynamic increase factors:

Concrete flexure:	1.19
diagonal tension:	1.00
direct shear:	1.10
Prestressing Steel:	1.00
Welding Wire Fabric:	1.10
Reinforcing Steel flexure:	1.17
shear:	1.00

b. Dynamic strengths.

Concrete

- flexure: f'_{dc} = 1.19 X 5,000 = 5950 psi
- diagonal tension: f'_{dc} = 1.0 X 5,000 = 5000 psi
- direct shear: f'_{dc} = 1.1 X 5,000 = 5500 psi

Prestressing Steel: f_{pu} = 1.0 X 270,000 = 270,000 psi

Welded Wire Fabric: f_{dy} = 1.10 X 1.10 X 65,000 = 78,650 psi

Reinforcing Bars

- flexure: f_{dy} = 1.17 X 1.10 X 60,000 = 77,220 psi
- shear: f_{dy} = 1.0 X 1.10 X 60,000 = 66,000 psi

Step 4. Calculate modulus of elasticity and modular ratio.

a. Concrete

$$E_c = 33w_c^{1.5}(f'_c)^{1/2}$$

$$= 33(150)^{1.5}(5000)^{1/2}$$

$$= 4.29 \times 10^6 \text{ psi}$$

b. Steel

$$E_s = 29.0 \times 10^6 \text{ psi}$$

c. $$n = \frac{E_s}{E_c} = \frac{29.0 \times 10^6}{4.29 \times 10^6} = 6.76$$

NOTE: The flange is designed in accordance with the principles of Volume IV. Considering the flange a one-way non-prestressed slab, the cantilever portion was found to be critical. In order to remain elastic the flange thickness must be increased to 3 inches. The reinforcement is two layers of welded wire fabric; 6 X 6 - W1.4 X W2.0 in the top and 6 X 6-W1.4 X W1.4 in the bottom.

Step 5. Calculate the section properties of the double tee with a 3 inch flange.

a. new $A = 497 \text{ in}^2$

b. new $y_t = 6.43$ in

new $I_g = 25180 \text{ in}^4$

c. $w = 497 \times 150 \text{ pcf}/12^3 = 43.1$ lb/in

d. $d_p = 7.85 + 14.15 = 22.0$ in

e. $A_{ps} = 4 \times 0.153 = 0.612 \text{ in}^2$

f. Prestressed reinforcement ratio (eq. 6-23)

$$p_p = A_{ps}/bd_p = 0.612/(96 \times 22.0) = 0.000290$$

Step 6. Determine the average stress in the prestressing tendon.

a. $f_{py}/f_{pu} = 0.85 \therefore \gamma_p = 0.40$

b. $\beta_1 = 0.85 - 0.05(5950-4000)/1000 = 0.7525$

c. Average stress (eq. 6-27)

$$f_{ps} = f_{pu}[1-\frac{\gamma_p}{\beta_1}(p_p \frac{f_{pu}}{f'_{dc}})]$$

$$= 270{,}000 \left[1 - \frac{0.40}{0.7525}\left(0.000290 \times \frac{270{,}000}{5950}\right)\right]$$

$$f_{ps} = 268{,}111 \text{ psi}$$

Step 7. Check maximum reinforcement ratio (eq. 6-30)

$$p_p f_{ps}/f'_{dc} \le 0.36\beta_1$$

$$p_p f_{ps}/f'_{dc} = 0.000290 \times 268{,}111/5950$$

$$= 0.0131$$

$$0.36\beta_1 = 0.36 \times 0.7525$$

$$= 0.2709 > 0.0131 \qquad \text{O.K.}$$

Step 8. Calculate moment capacity of beam.

From equation 6-21

$$a = \frac{A_{ps}f_{ps}}{0.85f'_{dc}b} = \frac{0.612 \times 268{,}111}{0.85 \times 5950 \times 96} = 0.34 \text{ in}$$

$c = a/\beta_1 = 0.34/0.7525 = 0.45 < 3.0$ in thick flange

Hence the neutral axis is within the flange and the section can be analyzed as a rectangular section. If the neutral axis had extended into the web, a strain compatibility analysis would be required.

From equation 6-20

$$M_u = A_{ps}f_{ps}(d_p - a/2)$$

$$= 0.612 \times 268.111 \ (22.0 - 0.34/2)$$

$$M_u = 3582 \text{ k-in}$$

Step 9. Find the resistance available to resist blast load.

a. Find the ultimate resistance (table 3-1).

$$r_u = 8M_u/L^2$$

$$= 8 \times 3582/480^2 = 0.124 \text{ k/in}$$

$$= 124 \text{ lbs/in}$$

b. Resistance available for blast load

$$r_{avail} = r_u - DL - LL$$

$$= 124 - 43.1 - (15 \text{ psf} \times 8 \text{ ft}/12)$$

$$= 124 - 43.1 - 10.0$$

$$= 70.9 \text{ lb/in}$$

Step 10. Determine the average moment of inertia.

a. Moment of inertia of cracked section (eq. 6-33).

$$I_c = nA_{ps}d_p^2[1-(p_p)^{1/2}]$$

$$= 6.76 \times 0.612(22.0)^2 \ [1-(0.00029)^{1/2}]$$

$$= 1970 \text{ in}^4$$

b. Average moment of inertia (eq. 6-32)

$$I_a = (I_g + I_c)/2$$

$$= (25180 + 1970)/2$$

$$= 13575 \text{ in}^4$$

Step 11. Using equations of table 3-8, calculate the elastic stiffness.

$$K_E = \frac{384 E_c I_a}{5L^4} = \frac{384 \times 4.29 \times 10^6 \times 13575}{5 \times 480^4}$$

$$= 84.25 \text{ lb/in/in}$$

Step 12. Calculate the effective mass

a. Load-mass factor (table 3-12)

In the elastic range

$$K_{LM} = 0.78$$

b. Unit mass

$$m = w/g = 43.1/(32.2 \times 12)$$

$$= 0.1116 \text{ lb-s}^2/\text{in}^2$$

$$= 11.16 \times 10^4 \text{ lb-ms}^2/\text{in}^2$$

c. Effective mass

$$m_e = K_{LM} m$$

$$= 0.78 \times 11.16 \times 10^4$$

$$= 8.70 \times 10^4 \text{ lb-ms}^2/\text{in}^2$$

Step 13. Calculate the natural period of vibration (eq. 3-60).

$$T_N = 2\pi(m_e/K_E)^{1/2}$$

$$= 2\pi(8.70 \times 10^4/84.25)^{1/2}$$

$$= 201.9 \text{ ms}$$

Step 14. Determine response of beam.

T = duration of load = 43.9 ms (step 1a)

T_N = natural period = 201.9 ms (step 13)

$T/T_N = 43.9/201.9 = 0.217$

From figure 3-49

$DLF = 0.65$

Actual resistance obtained $r = DLF \times P \leq r_{avail}$

P = peak dynamic load

= 1.1 psi (step 1a)

$r = 0.65 \times (1.1 \times 96)$

$= 68.4$ lb/in < 70.9 lb/in (r_{avail}, step 9) O.K.

Step 15. Check rotation.

a. Total load on beam = (DLF X p) + DL + LL

= 68.4 + 43.1 + 10.0

= 121.5 lb/in

b. Maximum deflection:

$X_m = (121.5 \text{ lb/in})/K_E$

$= 121.5/84.25$

$= 1.44$ in

c. Support rotation (table 3-5)

$\theta = \tan^{-1}(2X_m/L)$

$= \tan^{-1}(2 \times 1.44/480)$

$= 0.34° < 2°$ O.K.

Step 16. Calculate percent of rebound.

a. Calculate elastic deflection (eq. 3-36)

$X_E = r_u/K_E$

$= 124/84.25$

$= 1.47$ in

b. Calculate ductility ratio.

$$\mu = X_m/X_E$$

$$= 1.44/1.47$$

$$= 0.98$$

c. Find percent of rebound from figure 3-268.

$X_m/X_E = 0.98$ and $T/T_N = 0.216$

$r^-/r = 1.0$

$\therefore$ 100 percent rebound

Step 17. Determine required rebound moment capacity.

a. Required rebound resistance

$$r^-_{req.} = r^- - DL > r_{avail}/2$$

$$= 68.4 - 43.1$$

$$= 25.3 \text{ lb/in}$$

$r_{avail}/2 = 70.9/2 = 35.4 > 25.3$ lb/in

$\therefore$ Use 35.4 lb/in or considering a single tee

$r^-_{req} = 17.7$ lb/in

b. Required moment capacity (table 3-1).

$$M^-_u = r^-_{req}L^2/8$$

$$= 17.7 \times 480^2/8$$

$$M^-_u = 509{,}760 \text{ in-lb/stem}$$

Step 18. Determine rebound reinforcement.

a. Approximate value of d^-

$$d^- = h - \text{cover} - \Phi_{wire} - \Phi_{tie} - \frac{\Phi_{bar}}{2}$$

$$= 25 - 0.625 - 0.135 - 0.375 - 0.5/2$$

$$= 23.62 \text{ in}$$

b. Rebound concrete strength

$f = 0.47f'_{dc}$

$= 0.47 \times 5950$

$= 2796.5$ psi

c. Required rebound reinforcement

Assume a = 2.0 in

$A_s^- = M_u^-/[f_{dy}(d^- - a/2]$ (eq. 6-34)

$= 509{,}760/[77{,}220(23.62 - 2.0/2)]$

$= 0.29\ in^2$

$$a = \frac{A_s^- f_{dy}}{(0.47f'_{dc})b}$$

$$= \frac{0.29 \times 77{,}220}{2796.5 \times 3.75} = 2.1 \text{ in} \doteq 2.0 \text{ in} \qquad \text{O.K.}$$

Use 2 No. 4 bars in each stem

$A_s^- = 0.40\ in^2$

d. Check maximum reinforcement (eq. 6-36).

$$A_s^- \leq \frac{0.47f'_{dc}\beta_1}{f_{dy}} \left[\frac{87{,}000 - 0.378nf'_{dc}}{(87{,}000 - 0.378nf'_{dc} + f_{dy})}\right] bd^-$$

$$= \frac{2796.5 \times 0.7525}{77{,}220} \left[\frac{87{,}000-0.378 \times 6.76 \times 5950}{(87{,}000-0.378 \times 6.76 \times 5950 + 77{,}220)}\right]$$

$\times (3.75 \times 23.62)$

$= 1.16\ in^2 > 0.40\ in^2$ O.K.

Step 19. Design the shear reinforcement.

Calculate shear at distance d_p from support.

$v_u = r(L/2 - d_p)/b_w d_p$

r = 121.5 lb/in (total load on beam, step 15a)

$v_u = 121.5\ (480/2 - 22.0)/(2 \times 4.75 \times 22.0)$

$= 127$ psi

b. Maximum allowable shear stress.

$10\,(f'_c)^{1/2} = 10 \times (5000)^{1/2}$

$= 707$ psi > 127 psi   O.K.

c. Allowable shear stress on unreinforced web.

$v_c = 1.9(f'_c)^{1/2} + 2500p_p \leq 2.28(f'_c)^{1/2}$

$$= 1.9(5000)^{1/2} + 2500\left(\frac{0.612}{2 \times 4.75 \times 22}\right)$$

$= 142$ psi

$2.28(f'_c)^{1/2} = 2.28(5000)^{1/2}$

$= 161$ psi > 142 psi   O.K.

d. Excess shear.

$v_u - v_c \geq v_c$

$v_u - v_c = 127 - 142 = -15$ psi

$\therefore$ use v_c

e. Shear reinforcement.

Assume #3 closed ties

$A_v = 2 \times 0.11 = 0.22\ in^2$

$$s_s = \frac{A_v \Phi f_y}{v_c b} = \frac{0.22 \times 0.85 \times 66{,}000}{142 \times 4.75}$$

$= 18.3$ in

f. Check maximum spacing and miminum required reinforcement.

$s \leq d_p/2 = 22.0/2 = 11.0$ in.

$A_v \geq 0.0015bs_s$

$= 0.0015 \times 4.75 \times 11.0$

$= 0.08\ in^2 > 0.22\ in^2$

Shear reinforcement is thus #3 ties at 11 inches in both stems.

Step 20. Check direct shear.

a. Calculate shear at the support (table 3-9)

$V_d = rL/2$

$= 121.5 \times 480/2$

$= 29160$ lbs

b. Calculate allowable direct shear.

$V_d \leq 0.18 f'_c bd$

$= 0.18 \times 5{,}500 \times (2 \times 4.75) \times 22.0$

$= 206{,}910$ lbs $> 29{,}160$ lbs O.K.

Step 21. Check section for conventional loads.

This section as designed for blast loads is shown in figure 6A-3. Using the PCI Design Handbook and the latest ACI code, the section must be checked to make sure it is adequate for service loads.

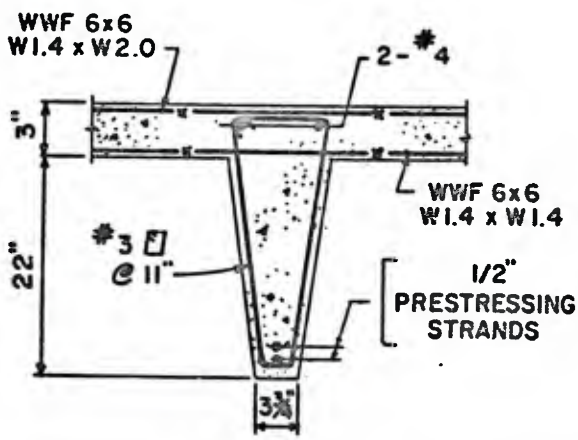


Figure 6A-3

Problem 6A-3 Design of Windows

Problem: Determine the minimum thickness of glazing to resist a given blast load, and the design loads for the framing.

Procedure:

Step 1. Establish design parameters:

a. Pressure-time loading.

b. Dimensions of pane(s).

c. Type of glazing.

Step 2. Calculate aspect ratio oi pane.

Step 3. With the type of glazing (step 1c) and the aspect ratio of the pane (step 2) determine from table 6-6 which set of curves (from figure 6-28 through 6-42) apply. Using the peak pressure of the dynamic load, its duration and the short dimension of the pane, determeine the minimum required glazing thickness.

NOTE: If given window geometry differs from chart parameters, interpolation as outlined in Section 6-28.4 may be required.

Step 4. Find the static ultimate resistance r_u of the glazing from table 6-7 for the given aspect ratio, the short dimension and the thickness of the glazing (interpolate if required).

Step 5. From table 6-8 and the aspect ratio determine the design coefficients C_R, C_x and C_y for the window frame loading. With these coefficients, the dimensions of the pane from step 1b, the static ultimate resistance of step 4 and equations 6-44, 6-45 and 6-46, calculate the design load along the short span of the pane, along the long span and at the corners.

Example 6A-3 Design of Windows

Required: Find the minimum glazing thickness and the design loads on the frame of a non-operable window consisting of four equal size panes of glass.

Solution:

Step 1. Given:

a. Pressure-time loading (fig. 6A-4)

b. Each pane is 22.5 inches long by 18 inches high.

c. The glazing is heat-treated tempered glass meeting Federal Specification DO-G-1403B and ANSI Z97.1-1975.

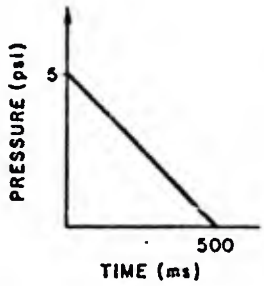


Figure 6A-4

Step 2. Calculate apsect ratio.

$a/b = 22.5/18 = 1.25$

Step 3. Determine minimum glazing thickness.

a. For $a/b = 1.25$ and tempered glass use figures 6-30 and 6-31 (table 6-1).

b. From figure 6-31b, when T = 500 ms and b = 18 in, a pane 3/16 inch thick can resist a blast load of 5.7 psi.

$P = 5.0$ psi < 5.7 psi

$\therefore$ t = 3/16 in. nominal.

Step 4. Find static ultimate resistance r_u, (table 6-7).

If $a/b = 1.25$

b = 18 inches

t = 3/16 inch

From table 6-7

$r_u = 9.18$ psi

The window frame must be designed to safely support, without undue deflections, a static uniform load of 9.18 psi applied normal to the glazing.

Step 5. Compute design loads on window frame.

a. Determine design coefficients from table 6-8, interpolating for $a/b = 1.25$.

$C_R = 0.077$

$C_x = 0.545$

$C_y = 0.543$

b. Calculate the unit shear along the long span of the frame (eq. 6-44).

$V_x = C_x r_u b \sin(\Pi x/a)$

$= 0.545 \times 9.18 \times 18 \sin(\Pi x/22.5)$

$= 90.1 \sin(\Pi x/22.5)$ lb/in

c. Calculate the unit shear along the short span of the frame (eq. 6-45).

$V_y = C_y r_u b \sin(\Pi y/b)$

$= 0.543 \times 9.18 \times 18 \sin(\Pi y/18)$

$= 89.7 \sin(\Pi y/18)$ lb/in

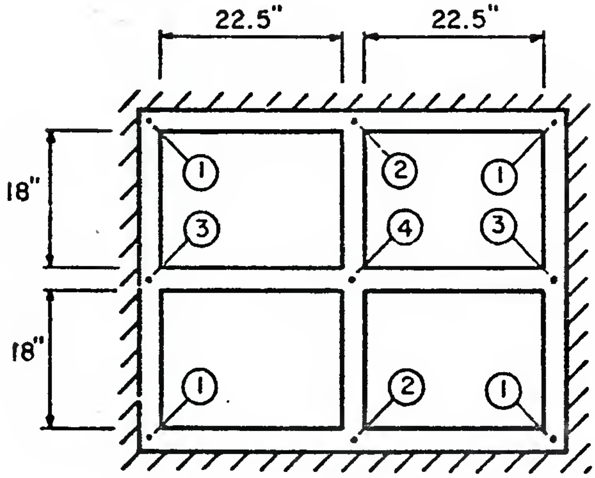
d. Calculate the uplift force at the corners of the panes (eq. 6-46).

$R_g = -C_R r_u b^2$

$= -0.077 \times 9.18 \times 18^2$

$= -229$ lbs.

The final design loads for the window frame are shown in figure 6A-5, below.



LOCATIONS	DESIGN LOAD
1	R
2 3	2R
4	4R
1 - 2	V_x
1 - 3	V_y
2 - 4	$2V_y$
3 - 4	$2V_x$

Fig. 6A-5

Problem 6A-4 Design of Shock Isolation System

Problem: Design an overhead pendulum shock isolation system using a platform for a given loading.

Step 1. Establish design parameters:

a. Structural configuration.
b. Magnitude and location of loads on platform.
c. Shock spectra for horizontal and vertical motion.
d. Maximum allowable motion.

Step 2. Compute member sizes of platform.

Step 3. Compute center of gravity of the loads (live and dead) on platform.

Step 4. Compute the elastic center of the spring supporting system.

Step 5. Determine the required weight and location of ballast to balance the system (i.e. to move the center of gravity of loads to coincide with the elastic center of the spring supporting system).

Step 6. Compute the total weight of the isolation system. Additional ballast equal to 25% of the weight of the equipment and ballast from Step 5 is added to provide for future changes in equipment. Also determine the equivalent uniform load equal to the total load divided by the area of the platform.

Step 7. Determine the natural frequency of the individual members of the platform (Using the equivalent uniform load computed in Step 6). The natural frequency is

$$f = \frac{9.87}{2\pi} \quad \frac{EIg}{wL^4}$$

for a simply supported beam with a uniform load.

Step 8. Using the shock spectrum for vertical motion, determine the required frequency of the system that will reduce the input accelerations to the maximum alowable. In addition, determine the displacement at this frequency.

Step 9. Verify rigid body motion of the platform.

The natural frequency of the individual members of the platform should be at least 5 times greater than the natural frequency of the system for rigid body motion of the platform to occur.

To increase the frequency of the individual members, increase member sizes, and repeat Steps 3 through 7.

Step 10. Determine the natural frequency for horizontal motion (pendulum action) of the platform where

$$f_{horizontal\ motion} = \frac{1}{2\pi}\left(\frac{386.4}{L}\right)^{1/2} \qquad (6\text{-}56)$$

where L is the suspended length.

Step 11. Verify that dynamic coupling will not occur between vertical and horizontal motions. According to Section 6-47.3.2 dynamic coupling will not occur if

$$f_{horizontal\ motion} < \tfrac{1}{2} f_{vertical\ motion}$$

Step 12. From the shock spectra for horizontal motion, determine the maximum dynamic displacement, velocity and acceleration using the frequency computed in Step 10.

Verify that the maximum acceleration is less than the allowable.

Step 13. Compute the load in each spring.

$$\text{Load in each spring} = \frac{\text{Total Load}}{\text{Number of Springs}}$$

Step 14. Determine stiffness of springs to produce the required frequency of the system. Compute the static and maximum displacement. Using the static displacement and the load in each spring from Step 13, calculate the required spring stiffness K from,

$$K = \frac{\text{load in each spring}}{\text{static displacement}}$$

Or using the vertical frequency from Step 8, and the mass on each spring the stiffness can be calculated from

$$K = (2\pi f_{vert})^2 m$$

In addition, compute the travel of the spring according to Section 6-48.2, i.e.

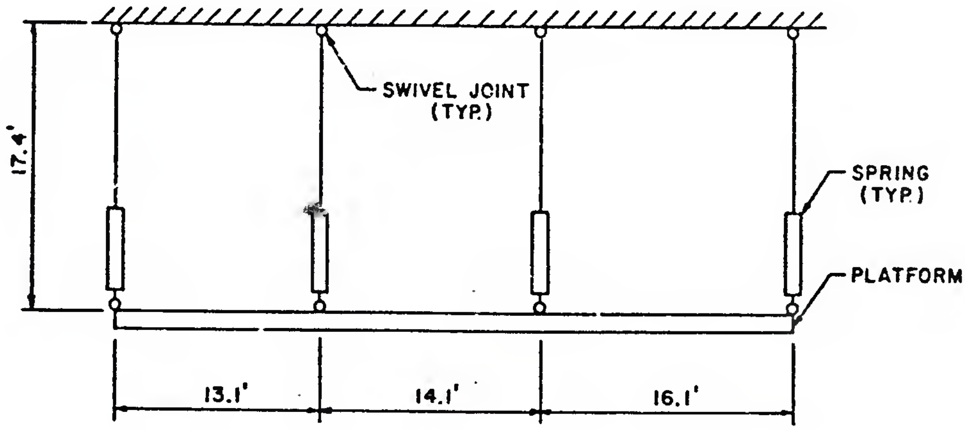
$$\text{travel} = \frac{\text{Maximum displacement}}{0.85}$$

Example 6A-4 Design Shock Isolation System

Required: Design an overhead pendulum shock isolation system using a platform for a given loading.

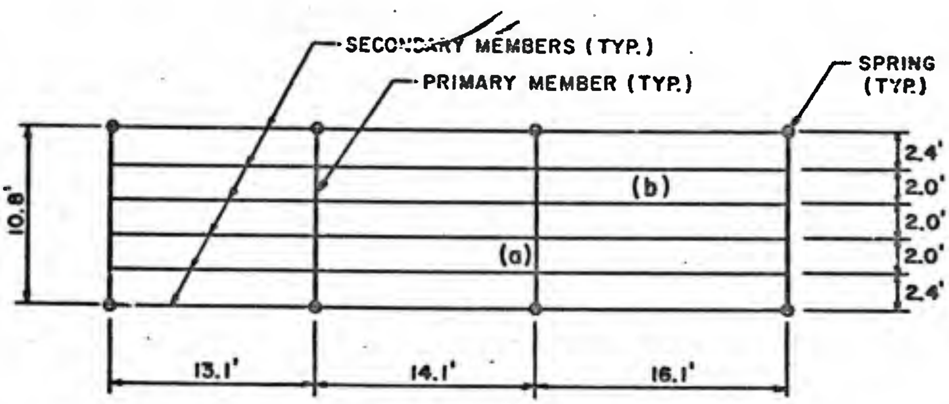
Step 1. Given:

a. Structural configuration shown in figure 6A-6a and figure 6A-6b.



ELEVATION

Fig. 6A - 6b



PLATFORM (PLAN)

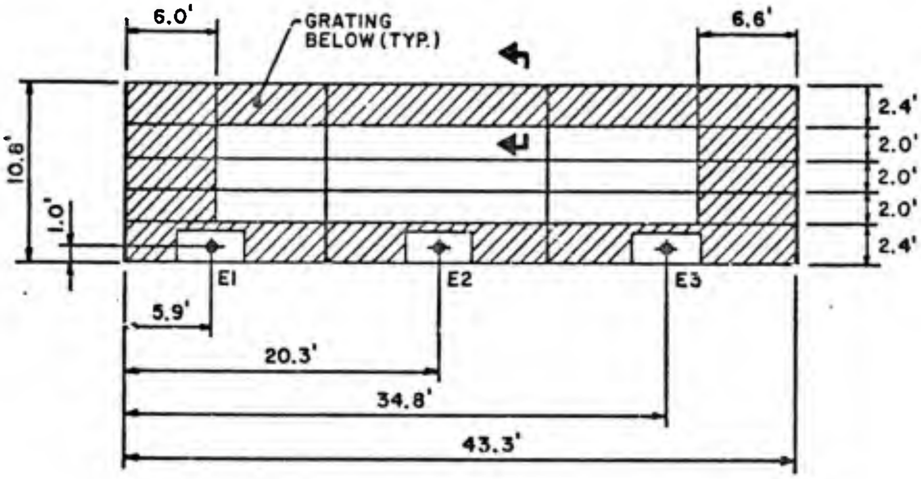
Fig. 6A - 6a

b. Magnitude and location of loads on platform (figure 6A-7)

Equipment: E_1 = 4960 lbs.
E_2 = 4960 lbs.
E_3 = 4960 lbs.

Floor panel = 5.5 psf (covers the whole area of platform)

Grating = 7.2 psf
Live load = 150 psf



PLATFORM (PLAN)

SECTION

Figure 6A-7

c. Shock spectra for horizontal and verticle motions are given in figure 6A-8a and 6A-8b, respectively.

d. Maximum allowable acceleration

Vertical Motion = 0.5g

Horizontal Motion = 0.75g

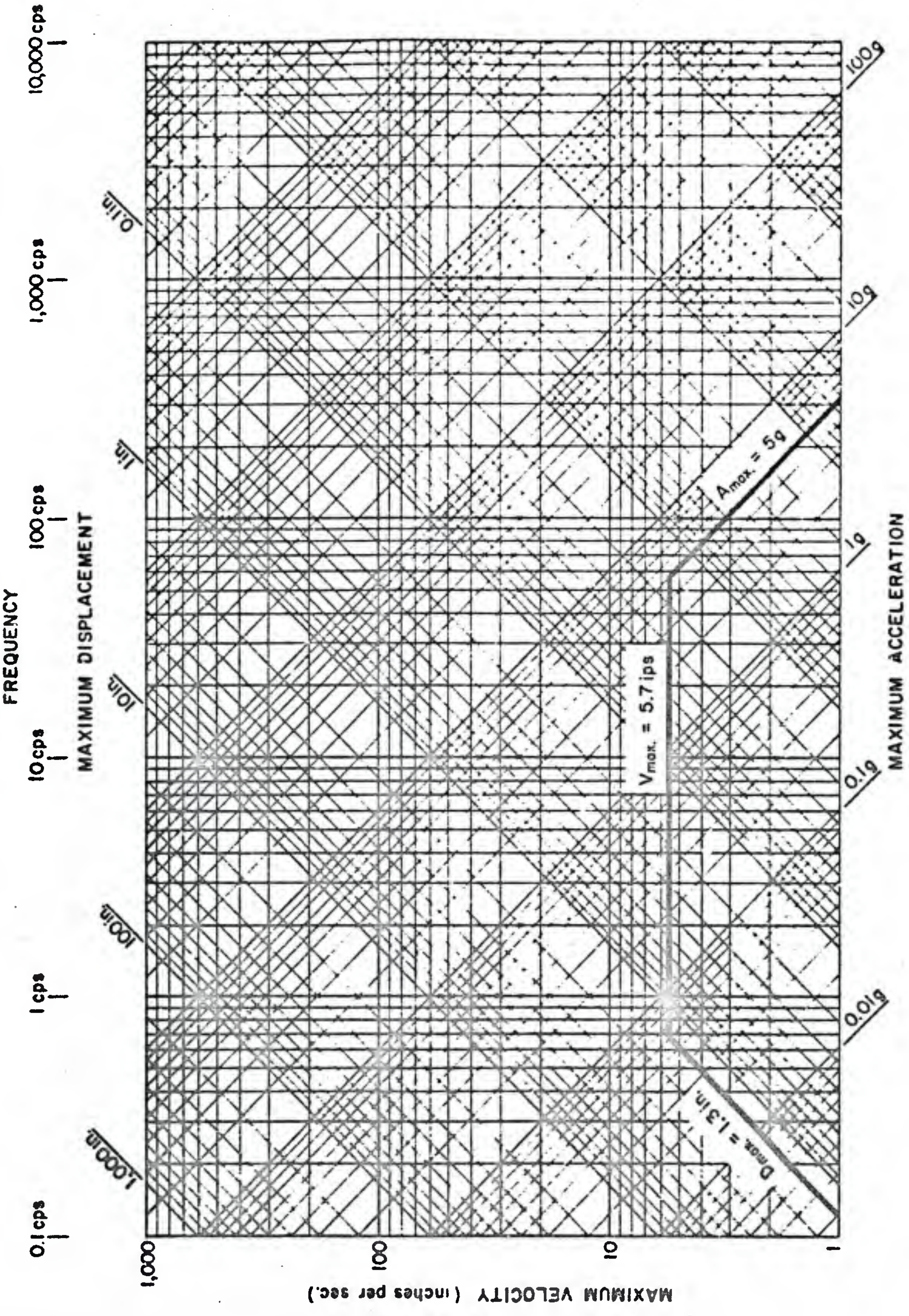

FREQUENCY
0.1 cps
1 cps
10 cps
100 cps
1,000 cps
10,000 cps
MAXIMUM DISPLACEMENT
1,000 in.
100 in.
10 in.
1 in.
0.1 in.
MAXIMUM ACCELERATION
0.01g
0.1g
1g
10g
100g
MAXIMUM VELOCITY (inches per sec.)
1,000
100
10
1
$A_{max.} = 5g$
$V_{max.} = 5.7 ips$
$D_{max.} = 1.3 in.$

Fig. 6A-8a

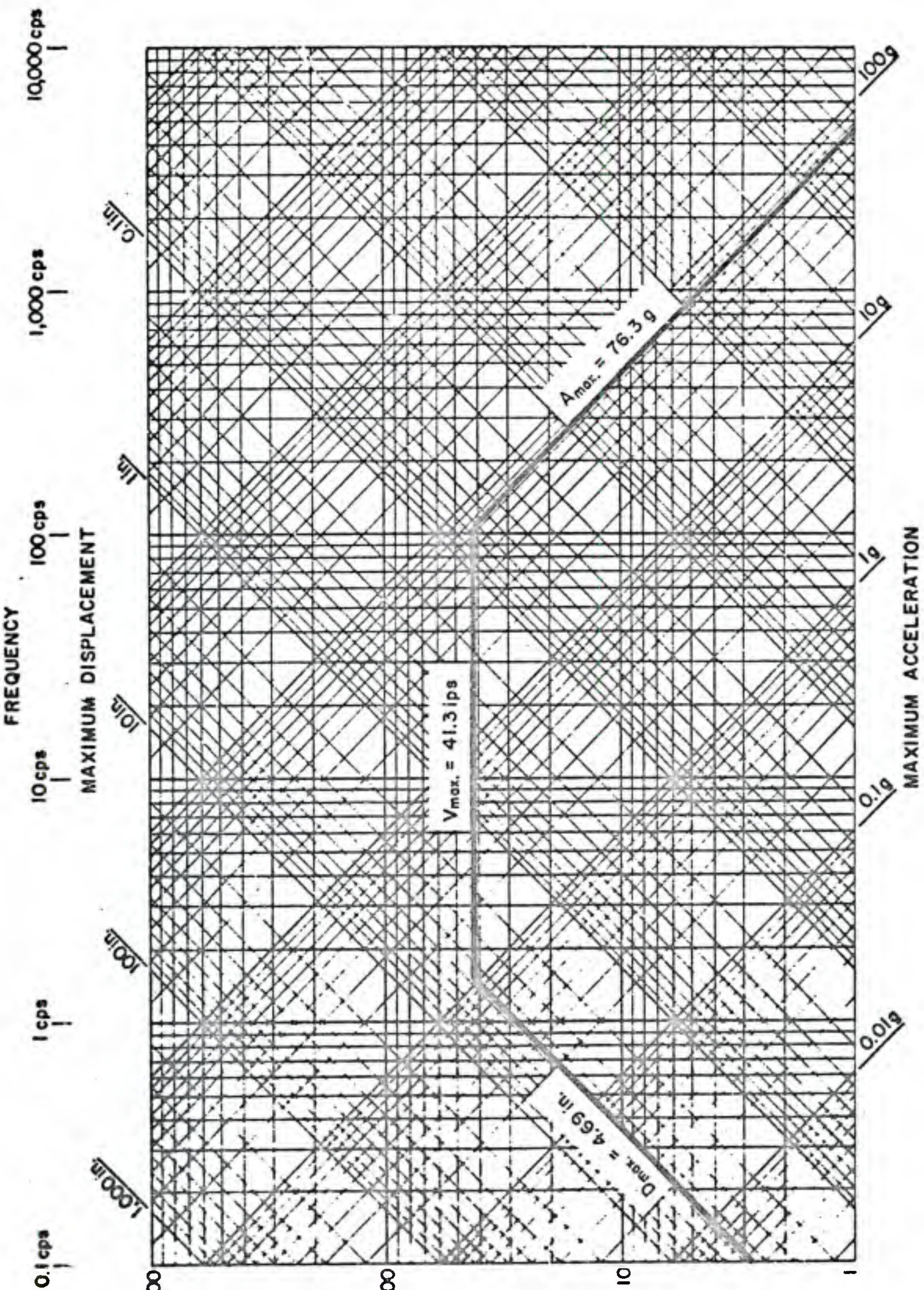

FREQUENCY
0.1 cps
1 cps
10 cps
100 cps
1,000 cps
10,000 cps
MAXIMUM DISPLACEMENT
1,000 in.
100 in.
10 in.
1 in.
0.1 in.
MAXIMUM ACCELERATION
100g
10g
1g
0.1g
0.01g
MAXIMUM VELOCITY (inches per sec.)
1,000
100
10
1
$A_{max.}$ = 76.3 g
$V_{max.}$ = 41.3 ips
$D_{max.}$ = 4.69 in.

Fig. 6A-8b

Step 2. Member sizes

Design Loads:

Dead load = 20 psf (assume)
Live Load = 150 psf

Primary Member (a): - (See fig. 6A-6a).

$$w = 170 \times (7.05 + 8.05)$$
$$= 2567 \text{ lbs/ft.}$$

Maximum Bending moment $= \dfrac{wL^2}{8}$ (at center)

$$= \frac{2567 \times 10.8^2}{8}$$

$$= 37427 \text{ lb.-ft.}$$

Using allowable stress design,

Allowable bending stress $= 0.66F_y$ (for compact shapes AISC).

$\therefore$ Required $S_x = \dfrac{\text{Maximum Bending Moment}}{\text{Allowable Bending Stress}}$

$$= \frac{37427 \times 12}{0.66 \times 36{,}000}$$

$$= 18.9 \text{ in}^3$$

Try section W10 X 21,

$$S_x = 21.5 \text{ in}^3 > 18.9 \text{ in}^3 \quad \text{O.K.}$$

Check deflection:

Maximum allowable deflection $= \dfrac{L}{360}$

$$= \frac{10.8 \times 12}{360}$$

Maximum deflection $= \dfrac{5WL^4}{384EI}$ (at center)

For W10 X 21,

$$I = 107 \text{ in}^3$$

Maximum deflection $= \dfrac{5 \times (2567/12) \times (10.8 \times 12)^4}{384 \times 29{,}000 \times 10^3 \times 107}$

$$= 0.25 \text{ in} < 0.36 \text{ in.} \quad \text{O.K.}$$

$\therefore$ Use W10 X 21 for all primary members.

Secondary Member (b): See fig. 6A-6a.

$$w = 170\ (1.0 + 1.2)$$
$$= 374 \text{ lb/ft.}$$

Maximum Bending moment $= \dfrac{wL^2}{8}$ (at center)

$$= \frac{374 \times 16.1^2}{8}$$

$$= 12118 \text{ lb. ft.}$$

Using allowable stress design,

Allowable bending stress = 0.66 F_y (for compact shapes, AISC).

$\therefore$ Required $S_x = \dfrac{12118 \times 12}{0.66 \times 36{,}000} = 6.12 \text{ in}^3$

Try section W8 x 13,

$$S_x = 9.90 \text{ in}^3 > 6.12 \text{ in}^3 \quad \text{O.K.}$$

Check deflection:

Maximum allowable deflection $= \dfrac{L}{360}$

$$= \frac{16.1 \times 12}{360}$$

$$= 0.54 \text{ in.}$$

For W8 X 13,

$$I = 39.6 \text{ in}^4$$

Maximum deflection $= \dfrac{5WL^4}{384\ EI}$

$$= \frac{5 \times (374/12) \times (16.1 \times 12)^4}{384 \times 29{,}000 \times 10^3 \times 39.6}$$

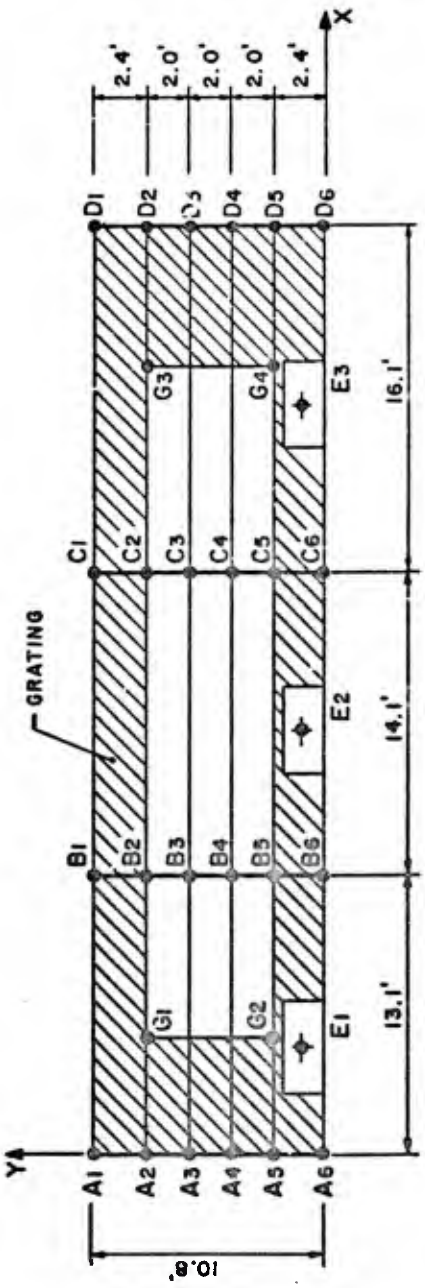
$$= 0.49 < 0.54 \quad \text{O.K.}$$

$\therefore$ Use W8 X 13 for all secondary members.

<u>NOTE:</u> Members should be checked for concentrated equipment loads.

Step 3. Find center of gravity of the loads on the platform (see Figure 6A-9).

Item	Weight W_1 (lbs.)	X (ft.)	Y (ft.)	W_1X (lb. ft.)	W_1Y (lb. ft.)
$A_1A_2 \ldots A_6$	226.8	0.0	5.4	0.0	1224.7
$B_1B_2 \ldots B_6$	226.8	13.1	5.4	2971.1	1224.7
$C_1C_2 \ldots C_6$	226.8	27.2	5.4	6169.0	1224.7
$D_1D_2 \ldots D_6$	226.8	43.3	5.4	9820.4	1224.7
$A_1B_1C_1D_1$	562.9	21.65	10.8	12187.0	6079.3
$A_2B_2C_2D_2$	562.9	21.65	8.4	12187.0	4728.4
$A_3B_3C_3D_3$	562.9	21.65	6.4	12187.0	3602.6
$A_4B_4C_4D_4$	562.9	21.65	4.4	12187.0	2476.8
$A_5B_5C_5D_5$	562.9	21.65	2.4	12187.0	1351.0
$A_6B_6C_6D_6$	562.9	21.65	0.0	12187.0	0.0
E_1	4960.0	5.9	1.0	29264.0	4960.0
E_2	4960.0	20.3	1.0	100688.0	4960.0
E_3	4960.0	34.8	1.0	172608.0	4960.0
$A_1D_1D_2A_2$	748.2	21.65	9.6	16199.0	7183.0
$A_5D_5D_6A_6$	748.2	21.65	1.2	16199.0	897.8
$A_2G_{11}G_{12}A_5$	259.2	3.0	5.4	777.6	1400.0
$D_2D_5G_{14}G_{15}$	259.2	4.0	5.4	10368.0	1400.0
Floor Panel	2572.0	21.65	5.4	55684.0	13889.0
Σ	23751.4		Σ	493870.0	62786.7



X
Y
GRATING
2.4'
2.0'
2.0'
2.0'
2.4'
D1
D2
D3
D4
D5
D6
C1
C2
C3
C4
C5
C6
B1
B2
B3
B4
B5
B6
A1
A2
A3
A4
A5
A6
G1
G2
G3
G4
E1
E2
E3
16.1'
14.1'
13.1'
10.8'

Fig. 6A-9

Center of gravity of loads (dead + live) = $\{\bar{X}_1, \bar{Y}_1\}$

Then, $\bar{X}_1 = \dfrac{\sum W_1 X}{\sum W_1}$

$= \dfrac{493870}{23751.4}$

$= 20.8$ ft.

$Y_1 = \dfrac{\Sigma W_1 Y}{\Sigma W}$

$= \dfrac{62786.7}{23751.4}$

$= 2.64$ ft.

Step 4. Elastic center of the spring support system.

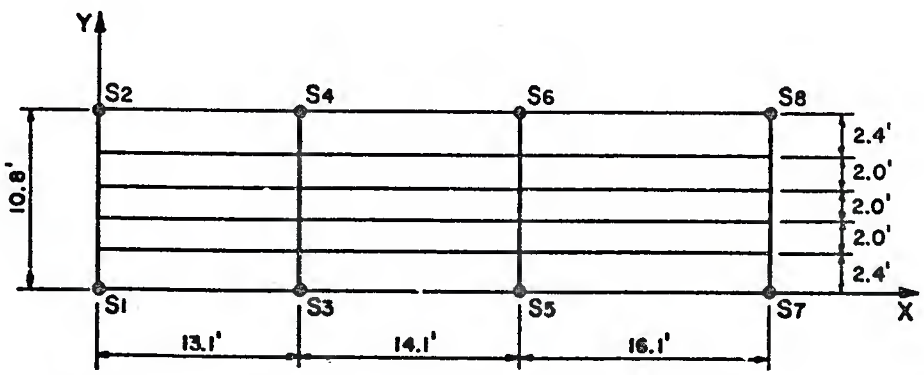


Figure 6A-10

NOTE: All springs have the same stiffness.

Spring No.	Force (W)	X (ft.)	Y (ft.)	WX	WY
S_1	P	0	0	0	0
S_2	P	0	10.8	0	10.8P
S_3	P	13.1	0	13.1P	0
S_4	P	13.1	10.8	13.1P	10.8P
S_5	P	27.2	0	27.2P	0
S_6	P	27.2	10.8	27.2P	10.8P
S_7	P	43.3	0	43.3P	0
S_8	P	43.3	10.8	43.3P	10.8P
$\sum$	8P		$\sum$	167.2P	43.2P

Elastic center of spring support system = $\{\bar{X}_s, \bar{Y}_s\}$.

$$\bar{X}_s = \frac{\sum WX}{\sum W}$$

$$= \frac{167.2P}{8P}$$

$$= 20.9 \text{ ft.}$$

$$\bar{X}_s = \frac{\sum WY}{\sum W}$$

$$= \frac{43.2P}{8P}$$

$$= 5.4 \text{ ft.}$$

Step 5. Find the weight and location of ballast to balance the system (relocate the c.g. of the platform to coincide with the elastic center of the isolation system).

a. For the x direction - Try placing ballast at x = 39.3 ft., i.e. 4 feet from the right edge of the platform. The ballast is placed symmetrically about the x axis of the elastic center so

as not to affect the location of the center of gravity in the y direction.

Weight of ballast = W_{BX}

$$\sum W_1 X + 39.3 W_{BX} = \bar{X}_s(\sum W_1 + W_{BX})$$

$$493870 + 39.3 W_{BX} = 20.9(23751.4 + W_{BX})$$

$$W_{BX} = 2534.3/18.4$$

$$= 137.7 \text{ lbs.}$$

b. For the y direction - Try placing ballast at y = 9.6 feet, i.e. 1.2 feet from the top of the platform. The ballast is placed symmetrically about the y axis of the elastic center so as not to affect the location of the center of gravity in the x direction.

Weight of ballast = W_{BY}

$$\sum W_1 Y + 9.6 W_{BY} = \bar{Y}_s(\sum W_1 + W_{BY})$$

$$62786.7 + 9.6 W_{BY} = 5.4(2375.1 + W_{BY})$$

$$W_{BY} = 65470.9/4.2$$

$$= 15588.3 \text{ lbs.}$$

c. Total ballast

$$W_B = W_{BX} + W_{By}$$

$$= 137.7 + 15588.3$$

$$= 15726 \text{ lbs.}$$

Step 6. Total load on the platform and equivalent uniform load.

a. Total load.

Dead Load:

Floor panel	=	2572 lbs.
Members	=	4284.6
Grating	=	2014.8
$\sum$	=	8871.4 lbs.

Live Load:

Equipment	=	14880 lbs.
Ballast	=	15726
Additional Ballast = 0.25(30606)	=	7651.6
Personnel (5 @ 150 lb.)	=	750
$\sum$	=	39007.5 lbs.

Total load $= 8871.4 + 39007.5$

$= 47878.9$ lbs.

b. Equivalent uniform load

$$W = \frac{47878.9}{10.8 \times 43.3}$$

$= 102.4$ psf ≤ 170 psf O.K.

Preliminary design of platform members is O.K. However, members supporting ballast must be checked as their actual load may be higher than the equivalent uniform load.

Step 7. Natural frequency of the individual members of the platform.

For a simply supported member with a uniform load,

$$\text{Natural frequency } f_n = \frac{9.87}{2\pi} \sqrt{\frac{EIg}{WL^4}}$$

Primary Members (a): A portion of the adjacent slab acts with the beam. Add 20% of the mass of the slab on each side of the beam to the actual mass of the beam.

W10 X 21

$I = 107$ in^3.

$L = 10.8 \times 12$
$= 129.6$ inches

$b = 0.40(7.05 + 8.05)$
$= 6.04$ feet

$g = 386.4$ in/sec^2

$E = 29000 \times 10^3$ psi.

W: (Use equivalent uniform load).

$w = 102.4 \times 6.04$

$= 618.5$ lb/ft

$= 51.54$ lb/in

$$f_n = \frac{9.87}{2\Pi} \left(\frac{29000 \times 10^3 \times 107 \times 386.4}{51.54 \times (29.6)^4}\right)^{1/2}$$

$$= 14.3 \text{ cps}$$

Secondary Members (b): Since the spacing of the secndary beams is less than 1/4 of the length of the beams the total mass of the slab act with the beam.

w 8 X 13 $I = 39.6 \text{ in}^4$

$L = 16.1 \times 12$
$= 193.2$ in.

$b = 2.2$ ft.

$w = 102.4 \times 2.2$
$= 225.3$ lb/ft
$= 18.78$ lb/in

$E = 29000 \times 10^3$ psi

$g = 386.4 \text{ in/sec}^2$

$$f_n = \frac{9.87}{2\Pi} \left(\frac{29,000 \times 10^3 \times 39.6 \times 386.4}{18.78 \times (193.2)^4}\right)^{1/2}$$

$$= 6.5 \text{ cps}$$

Step 8. Required frequency of system to limit motions.

To limit the maximum acceleration of the system to 0.5g or less, choose the frequency of the system from figure 6A-8b as

$$f = 1 \text{ cps}$$

which produces a maximum acceleration of 0.4g and a maximum dynamic displacement of 4.7 inches.

Step 9. Verification of rigid body motion of platform.

$$f_{\substack{\text{individual}\\ \text{member}}} > 5 \times f_{\text{system}}$$

From Step 8.,

$$f_{\text{system}} = 1 \text{ cps}$$

From Step 7.,

$$f_{\substack{primary\\member}} = 14.2 \text{ cps} > 5 \text{ cps} \qquad \text{O.K.}$$

$$f_{\substack{secondary\\member}} = 6.5 \text{ cps} > 5 \text{ cps} \qquad \text{O.K.}$$

Therefore, platform is in rigid body motion.

Step 10. Natural frequency of platform for horizontal motion (i.e. pendulum type action).

Assume the center of gravity of supported mass is located at the top of the platform so that the length of the pendulum arm is 17.4 feet.

Frequency of platform for horizontal motion

$$f = \frac{1}{2\pi} \quad \frac{386.4}{L} \qquad (6\text{-}56)$$

$$= \frac{1}{2\pi} \quad \frac{386.4}{17.4 \times 12}$$

$$= 0.22 \text{ cps}$$

Step 11. Check for dymanic coupling of vertical and horizontal motion.

From Step 8.,

$$f_{\text{vertical motion}} = 1 \text{ cps}$$

From Step 10.,

$$f_{\text{horizontal motion}} = 0.22 \text{ cps} < 1/2 \; f_{\text{vertical motion}} = 0.5 \text{ cps}$$

Dynamic coupling will not occur.

Step 12. Maximum dynamic displacement, velocity and acceleration for horizontal motion.

From shock spectra for horizontal motion, (fig. 6A-8a)
for $f = 0.22$ cps,

Maximum acceleration = 0.007g < 0.75g. O.K.

Maximum velocity = 0.9 in/sec.

Maximum dynamic displacement = 1.3 in.

The maximum dynamic displacement is the required "rattle space" or the minimum horizontal clearance between the platform and the structure or anything attached to the structure.

Step 13. Load in each spring.

$$\text{Load in each spring} = \frac{\text{Total Load}}{\text{Number of Springs}}$$

$$= \frac{47878.9}{8}$$

$$= 5985 \text{ lbs.}$$

Step 14. Design the springs.

a. Static displacement

From Step 8.,

For a maximum acceleration of 0.48g the maximum dynmic displacement = 4.7 in.

$\therefore$ Static displacement = 4.7/0.48

b. Stiffness of spring = 4.7/0.48

$$K = \frac{\text{Load in each spring}}{\text{Static displacement}}$$

$$= \frac{5985}{9.79} = 611.2 \text{ lb/in}$$

or

$$K = (2\Pi f_{vert})^2 m$$

$$= (2\Pi \times 1\text{cps})^2 (5985/386.4)$$

$$K = 611.5 \text{ lb/in}$$

c. Maximum travel of spring.

Maximum displacement = (static + dynamic) displacement

$$= 9.79 + 4.7$$

$$= 14.49 \text{ in}$$

From Section 6-48.2

$$\text{Travel of spring} = \frac{\text{maximum displacement}}{0.85}$$

$$= \frac{14.49}{0.85}$$

$$= 17.0 \text{ in.}$$

Thus the vertical rattle space (clearance) is 17.0 inches.

APPENDIX 6B - LIST OF SYMBOLS

a	(1)	acceleration (in./ms^2)
	(2)	depth of equivalent rectangular stress block (in.)
	(3)	long span of a panel (in.)
A		area (in.^{2})
A_a		area of diagonal bars at the support within a width b (in.^{2})
A_d		door area (in.^{2})
A_g		area of gross section (in^2)
A_n		net area of section (in.^{2})
A_o		area of openings (ft^2)
A_{ps}		area of prestressed reinforcement (in.^{2})
A_s		area of tension reinforcement within a width b (in.^{2})
A_s'		area of compression reinforcement within a width b (in.^{2})
A_s^-		area of rebound reinforcement (in.^{2})
A_{sH}		area of flexural reinforcement within a width b in the horizontal direction on each face (in.^{2})*
A_{sV}		area of flexural reinforcement within a width b in the vertical direction on each face (in.^{2})*
A_v		total area of stirrups or lacing reinforcement in tension within a distance, s_s or s_l and a width b_s or b_l (in.^{2}).
A_I, A_{II}		area of sector I and II, respectively (in.^{2})
b	(1)	width of compression face of flexural member (in.)
	(2)	width of concrete strip in which the direct shear stresses at the supports are resisted by diagonal bars (in.)
	(3)	short span of a panel (in.)

* See note at end of symbols.

b_s	width of concrete strip in which the diagonal tension stresses are resisted by stirrups of area A_v (in.)
b_l	width of concrete strip in which the diagonal tension stresses are resisted by lacing of area A_v (in.)
B	constant defined in pargraph
c	(1) distance from the resultant applied load to the axis of rotaton (in.) (2) damping coefficient (3) distance from extreme compression fiber to neutral axis (in.)
c_I, c_{II}	distance from the resultant applied load to the axis of rotation for sectors I and II, respectively (in.)
c_s	dilatational velocity of concrete (ft/sec)
C	shear coefficient
C_{cr}	critical damping
C_d	shear coefficient for ultimate shear stress of one-way elements
C_f	post-failure fragment coefficient (lb^2-$ms^4/in.^8$)
C_R	force coefficient for shear at the corners of a window frame
C_{ra}	peak reflected pressure coefficient at angle of incidence a
C_s	shear coefficient for ultimate support shear fcr one-way elements
C_{sH}	shear coefficient for ultimate support shear in horizontal direction for two-way elements*
C_{sV}	shear coefficient for ultimate support shear in vertical direction for two-way elements*
C_D	drag coefficient
C_{Dq}	drag pressure (psi)

* See note at end of symbols.

$C_D q_o$	peak drag pressure (psi)
C_E	equivalent load factor
C_H	shear coefficient for ultimate shear stress in horizontal direction for two-way elements*
C_L	leakage pressure coefficient
C_M	maximum shear coefficient
C_u	impulse coefficient at deflection X_u (psi-ms^2/in.^{2})
C_u'	impulse coefficient at deflection X_m (psi-ms^2/in.^{2})
C_v	shear coefficient for ultimate shear stress in vertical direction for two-way elements*
C_x	shear coefficient for the ultimate shear along the long side of window frame
C_y	shear coefficient for the ultimate shear along the short side of window frame
C_1	(1) impulse coefficient at deflection X_1 (psi-ms^2/in.^{2}) (2) parameter defined in figure (3) ratio of gas load to shock load
C_1'	impulse coefficient at deflection X_m (psi-ms^2/in.^{2})
C_2	ratio of gas load duration to shock load duration
d	distance from extreme compression fiber to centroid of tension reinforcement (in.)
d'	distance from extreme compression fiber to centroid of compression reinforcement (in.)
d_c	distance between the centroids of the compression and tension reinforcement (in.)

* See note at end of symbols.

d_{co}	diameter of steel core (in.)
d_e	distance from support and equal to distance d or d_c (in.)
d_i	inside diameter of cylindrical explosive container (in.)
d_l	distance between center lines of adjacent lacing bends measured normal to flexural reinforcement (in.)
d_p	distance from extreme compression fiber to centroid of prestressed reinforcement (in.)
d_1	diameter of cylindrical portion of primary fragment (in.)
D	(1) unit flexural rigidity (lb-in.)
	(2) location of shock front for maximum stress (ft)
	(3) minimum magazine separation distance (ft)
D_o	nominal diameter of reinforcing bar (in.)
D_E	equivalent loaded width of structure for non-planar wave front (ft)
DIF	dynamic increase factor
DLF	dynamic load factor
e	(1) base of natural logarithms and equal to 2.71828...
	(2) distance from centroid of section to centroid of prestressed reinforcement (in.)
$(2E')^{1/2}$	Gurney Energy Constant (ft/sec)
E	modulus of elasticity
E_c	modulus of elasticity of concrete (psi)
E_m	modulus of elasticity of masonry units (psi)
E_s	modulus of elasticity of reinforcement (psi)
f	(1) unit external force (psi)
	(2) frequency of vibration (cps)
f'_c	static ultimate compressive strength of concrete at 28 days (psi)

f'_{dc}	dynamic ultimate compressive strength of concrete (psi)
f'_{dm}	dynamic ultimate compressive strength of masonry units (psi)
f_{ds}	dynamic design stress for reinforcement (psi)
f_{du}	dynamic ultimate stress of reinforcement (psi)
f_{dy}	dynamic yield stress of reinforcement (psi)
f'_{m}	static ultimate compressive strength of masonry units (psi)
f_n	natural frequency of vibration (cps)
f_{ps}	average stress in the prestressed reinforcement at ultimate load (psi)
f_{pu}	specified tensile strength of prestressing tendon (psi)
f_{py}	yield stress of prestressing tendon corresponding to a 1 percent elongation (psi)
f_s	static design stress for reinforcement (a function of f_y, f_u and θ (psi)
f_{se}	effective stress in prestressed reinforcement after allowances for all prestress losses (psi)
f_u	static ultimate stress of reinforcement (psi)
f_y	static yield stress of reinforcement (psi)
F	(1) total external force (lbs)
	(2) coefficient for moment of inertia of cracked section
	(3) function of C_2 & C_1 for bilinear triangular load
F_o	force in the reinforcing bars (lbs)
F_E	equivalent external force (lbs)
g	(1) variable defined in table 4-3
	(2) acceleration due to gravity (ft/sec^2)
G	shear modulus (psi)

h (1) charge location parameter (ft)

(2) height of masonry wall

h' clear height between floor slab and roof slab

H (1) span height (in.)

(2) distance between reflecting surface(s) and/or free edge(s) in vertical direction (ft)

H_c height of charge above ground (ft)

H_c scaled height of charge above ground (ft/lb$^{1/3}$)

H_s height of structure (ft)

H_T scaled height of triple point (ft/lb$^{1/3}$)

i unit positive impulse (psi-ms)

i^- unit negative impulse (psi-ms)

$\bar{i}_a$ sum of scaled unit blast impulse capacity of receiver panel and scaled unit blast impulse attenuated through concrete and sand in a composite element (psi-ms/lb$^{1/3}$)

i_b unit blast impulse (psi-ms)

$\bar{i}_b$ scaled unit blast impulse (psi-ms/lb$^{1/3}$)

$\bar{i}_{bt}$ total scaled unit blast impulse capacity of composite element (psi-ms/lb$^{1/3}$)

$\bar{i}_{ba}$ scaled unit blast impulse capacity of receiver panel of composite element (psi-ms/lb$^{1/3}$)

$\bar{i}_{bd}$ scaled unit blast impulse capacity of donor panel of composite element (psi-ms/lb$^{1/3}$)

i_e unit excess blast impulse (psi-ms)

i_r unit positive normal reflected impulse (psi-ms)

i_r^- unit negative normal reflected impulse (psi-ms)

i_s	unit positive incident impulse (psi-ms)
i_{s^-}	unit negative incident impulse (psi-ms)
I	moment of inertia (in.^{4})
I_a	average of gross and cracked moments of inertia of width b (in.^{4})
I_c	moment of inertia of cracked concrete section of width b (in.^{4})
I_g	moment of inertia of gross concrete section of width b (in.^{4})
I_m	mass moment of inertia (lb-ms^2-in.)
I_n	moment of inertia of net section of masonry unit (in.^{4})
j	ratio of distance between centroids of compression and tension forces to the depth d
k	constant defined in paragraph
K	(1) unit stiffness (psi-in for slabs) (lb/in/in for beams)(lb/in for springs)
	(2) constant defined in paragraph
K_e	elastic unit stiffness (psi/in for slabs) (lb/in/in for beams)
K_{ep}	elasto-plastic unit stiffness (psi-in for slabs) (psi for beams)
K_E	equivalent elastic unit stiffness (psi-in for slabs) (psi for beams)
	equivalent spring constant
K_L	load factor
K_{LM}	load-mass factor
$(K_{LM})_u$	load-mass factor in the ultimate range
$(K_{LM})_{up}$	load-mass factor in the post-ultimate range
K_M	mass factor
K_R	resistance factor
K_1	factor defined in paragraph

KE	kinetic energy
l	charge location parameter (ft)
l_p	spacing of same type of lacing bar (in.)
L	(1) span length (in.)*
	(2) distance between reflecting surface(s) and/or free edge(s) in horizontal direction (ft)
L_1	length of lacing bar required in distance s_1 (in.)
L_o	embedment length of reinforcing bars (in.)
L_s	length of shaft (in.)
L_w	wave length of positive pressure phase (ft)
L_w^-	wave length of negative pressure phase (ft)
L_{wb}, L_{wd}	wave length of positive pressure phase at points b and d, respectively (ft)
L_1	total length of sector of element normal to axis of rotation (in.)
m	unit mass (psi-ms^2/in.)
m_a	average of the effective elastic and plastic unit masses (psi-ms^2/in.)
m_e	effective unit mass (psi-ms^2/in.)
m_u	effective unit mass in the ultimate range (psi-ms^2/in.)
m_{up}	effective unit mass in the post-ultimate range (psi-ms^2/in.)
M	(1) unit bending moment (in-lbs/in.)
	(2) total mass (lb-ms^2/in.)
M_e	effective total mass (lb-ms^2/in.)
M_u	ultimate unit resisting moment (in-lbs/in.)
M_u^-	ultimate unit rebound moment (in-lbs/in.)

* See note at end of symbols.

M_c moment of concentrated loads about line of rotation of sector (in.-lbs)

M_A fragment distribution parameter

M_E equivalent total mass ($lb\text{-}ms^2/in.$)

M_{HN} ultimate unit negative moment capacity in horizontal direction (in.-lbs/in.)*

M_{HP} ultimate unit positive moment capacity in horizontal direction (in.-lbs/in.)*

M_N ultimate unit negative moment capacity at supports (in.-lbs/in.)

M_P ultimate unit positive moment capacity at midspan (in.-lbs/in.)

M_{VN} ultimate unit negative moment capacity in vertical direction (in.-lbs/in.)*

M_{VP} ultimate unit positive moment capacity in vertical direction (in.-lbs/in.)*

n (1) modular ratio

(2) number of time intervals

(3) number of glass pane tests

N number of adjacent reflecting surfaces

N_f number of primary fragments larger than W_f

p reinforcement ratio equal to $\frac{A_s}{bd}$ or $\frac{A_s}{bd_c}$

p' reinforcement ratio equal to $\frac{A_s'}{bd}$ or $\frac{A_s'}{bd_c}$

p_b reinforcement ratio producing balanced conditions at ultimate strength

p_p prestressed reinforcement ratio equal to A_{ps}/bd_p

P_m mean pressure in a partially vented chamber (psi)

* See note at end of symbols.

p_{mo}	peak mean pressure in a partially vented chamber (psi)
p_H	reinforcement ratio in horizontal direction on each face*
p_T	reinforcement ratio equal to $p_H + p_V$
p_V	reinforcement ratio in vertical direction on each face*
$p(x)$	distributed load per unit length
P	(1) pressure (psi)
	(2) concentrated load (lbs)
$P-$	negative pressure (psi)
P_i	interior pressure within structure (psi)
P_i	interior pressure increment (psi)
P_f	fictitious peak pressure (psi)
P_o	peak pressure (psi)
P_r	peak positive normal reflected pressure (psi)
P_r-	peak negative normal reflected pressure (psi)
P_{ra}	peak reflected pressure at angle of incidence a (psi)
P_s	positive incident pressure (psi)
P_{sb}, P_{se}	positive incident pressure at points b and e, respectively (psi)
P_{so}	peak positive incident pressure (psi)
$P_{so}-$	peak negative incident pressure
P_{sob}, P_{sod}, P_{soe}	peak positive incident pressure at points b, d, and e, respectively (psi)
q	dynamic pressure (psi)
q_b, q_e	dynamic pressure at points b and e, respectively (psi)
q_o	peak dynamic pressure (psi)
q_{ob}, q_{oe}	peak dynamic presure at points b and e, respectively (psi)

* See note at end of symbols.

r	(1) unit resistance (psi)
	(2) radius of spherical TNT (density equals 95 lb/ft^3 charge (ft))
r^-	unit rebound resistance (psi, for panels) (lb/in for beams)
Δr	change in unit resistance (psi, for panels) (lb/in for beams)
r_d	radius from center of impulse load to center of door rotation (in.)
r_e	elastic unit resistance (psi, for panels) (lb/in for beams)
r_{ep}	elasto-plastic unit resistance (psi, for panels) (lb/in for beams)
r_s	radius of shaft (in.)
r_u	ultimate unit resistance (psi, for panels) (lb/in for beams)
r_{up}	post-ultimate unit resistant (psi)
r_1	radius of hemispherical portion of primary fragment (in.)
R	(1) total internal resistance (lbs)
	(2) slant distance (ft)
R_f	distance traveled by primary fragment (ft)
R_g	uplift force at corners of window frame (lbs)
R_l	radius of lacing bend (in.)
R_A	normal distance (ft)
R_E	equivalent total internal resistance (lbs)
R_G	ground distance (ft)
R_u	total ultimate resistance
R_I, R_{II}	total internal resistance of sectors I and II, respectively (lbs)
s	sample standard deviation
s_s	spacing of stirrups in the direction parallel to the longitudinal reinforcement (in.)

s_1	spacing of lacing in the direction parallel to the longitudinal reinforcement (in.)
S	height of front wall or one-half its width, whichever is smaller (ft)
SE	strain energy
t	time (ms)
Δt	time increment (ms)
t_a	any time (ms)
t_b, t_e, t_f	time of arrival of blast wave at points b, e, and f, respectively (ms)
t_c	(1) clearing time for reflected pressures (ms)
	(2) container thickness of explosive charges (in.)
t_d	rise time (ms)
t_E	time to reach maximum elastic deflection
t_m	time at which maximum deflection occurs (ms)
t_o	duration of positive phase of blast pressure (ms)
t_o^-	duration of negative phase of blast pressure (ms)
t_{of}	fictitious positive phase pressure duration (ms)
t_{of}^-	fictitious negative phase pressure duration (ms)
t_r	fictitious reflected pressure duration (ms)
t_u	time at which ultimate deflection occurs (ms)
t_y	time to reach yield (ms)
t_A	time of arrival of blast wave (ms)
t_1	time at which partial failure occurs (ms)
T	(1) duration of equivalent triangular loading function (ms)
	(2) thickness of masonry wall

T_c	thickness of concrete section (in.)
$\bar{T}_c$	scaled thickness of concrete section (ft/lb$^{1/3}$)
T_g	thickness of glass (in.)
T_i	angular impulse load (lb-ms-in.)
T_N	effective natural period of vibration (ms)
T_r	rise time (ms)
T_s	thickness of sand fill (in.)
$\bar{T}_s$	scaled thickness of sand fill (ft/lb$^{1/3}$)
u	particle velocity (ft/ms)
u_u	ultimate flexural or anchorage bond stress (psi)
U	shock front velocity (ft/ms)
U_s	strain energy
v	velocity (in./ms)
v_a	instantaneous velocity at any time (in./ms)
v_b	boundary velocity for primary fragments (ft/sec)
v_c	ultimate shear stress permitted on an unreinforced web (psi)
v_f	maximum post-failure fragment velocity (in./ms)
v_f (avg.)	average post-failure fragment velocity (in./ms)
v_i	velocity at incipient failure deflection (in./ms)
v_o	initial velocity of primary fragment (ft/sec)
v_r	residual velocity of primary fragment after perforation (ft/sec)
v_s	striking velocity of primary fragment (ft/sec)
v_u	ultimate shear stress (psi)
v_{uH}	ultimate shear stress at distance d_e from the horizontal support (psi)*

* See note at end of symbols.

v_{uV}	ultimate shear stress at distance d_e from the vertical support (psi)*
V	volume of partially vented chamber (ft^3)
V_d	ultimate direct shear capacity of the concrete of width b (lbs)
V_{dH}	shear at distance d_e from the vertical support on a unit width (lbs./in.)*
V_{dV}	shear at distance d_e from the horizontal support on a unit width (lbs/in.)*
V_o	volume of structure (ft^3)
V_s	shear at the support (lb/in, for panels) (lbs for beam)
V_{sH}	shear at the vertical support on a unit width (lbs/in.)*
V_{sV}	shear at the horizontal support on a unit width (lbs/in.)*
V_u	total shear on a width b (lbs)
V_x	unit shear along the long side of window frame (lb/in.)
V_y	unit shear along the short side of window frame, (lbs/in.)
w	unit weight (psi, for panels) (lb/in for beam)
w_c	weight density of concrete (lbs/ft^3)
w_s	weight density of sand (lbs/ft^3)
W	(1) charge weight (lbs)
	(2) weight (lbs)
W_c	total weight of explosive containers (lbs)
W_f	weight of primary fragment (oz)
W_{co}	total weight of steel core (lbs)
W_{c1}, W_{c2}	total weight of plates 1 and 2, respectively (lbs)
W_s	width of structure (ft)
WD	work done

x	yield line location in horizontal direction (in.)*
X	deflection (in.)
X_a	any deflection (in.)
X_c	lateral deflection to which a masonry wall develops no resistance (in.)
X_e	elastic deflection (in.)
X_{ep}	elasto-plastic deflection (in.)
X_f	maximum penetration into concrete of armor-piercing fragments (in.)
X_f'	maximum penetration into concrete of fragments other than armor-piercing (in.)
X_m	maximum transient deflection (in.)
X_p	plastic deflection (in.)
X_s	(1) maximum penetration into sand of armor-piercing fragments (in.)
	(2) static deflection
X_u	ultimate deflection (in.)
X_E	equivalent elastic deflection (in.)
X_1	(1) partial failure deflection (in.)
	(2) deflection at maximum ultimate resistance of masonry wall (in.)
y	yield line location in vertical direction (in.)*
y_t	distance from the top of section to centroid (in.)
Z	scaled slant distance (ft/lb$^{1/3}$)
Z_A	scaled normal distance (ft/lb$^{1/3}$)
Z_G	scaled ground distance (ft/lb$^{1/3}$)

* See note at end of symbols.

α	(1)	angle formed by the plane of stirrups, lacing, or diagonal reinforcement and the plane of the longitudinal reinforcement (deg)
	(2)	angle of incidence of the pressure front (deg)
	(3)	acceptance coefficient
β	(1)	coefficient for determining elastic and elasto-plastic resistances
	(2)	particular support rotation angle (deg)
	(3)	rejection coefficient
β_1		factor equal to 0.85 for concrete strengths up to 4000 psi and is reduced by 0.05 for each 1,000 psi in excess of 4,000 psi
γ		coefficient for determining elastic and elasto-plastic deflections
γ_p		factor for type of prestressing tendon
ϵ_m		unit strain in mortar (in./in.)
θ		support rotation angle (deg)
θ		angular acceleration (rad/ms^2)
θ_{max}		maximum support rotation angle (deg)
θ_H		horizontal rotation angle (deg)*
θ_V		vertical rotation angle (deg)*
λ		increase in support rotation angle after partial failure (deg)
μ		ductility factor
ν		Poisson's ratio
Σ_o		effective perimeter of reinforcing bars (in.)
ΣM		summation of moments (in.-lbs)
ΣM_N		sum of the ultimate unit resisting moments acting along the negative yield lines (in.-lbs)

* See note at end of symbols.

ΣM_p	sum of the ultimate unit resisting moments acting along the positive yield lines (in.-lbs)
τ_s	maximum shear stress in the shaft (psi)
ϕ	(1) capacity reduction factor
	(2) bar diameter (in.)
ϕ_r	assumed shape function for concentrated loads
$\phi(x)$	assumed shape function for distributed loads free edge
ω	angular velocity (rad./ms)
═══════	simple support
\\\\\\\\\\\\	fixed support
XXXXXXXX	either fixed, restrained, or simple support

* Note. This symbol was developed for two-way elements which are used as walls. When roof slabs or other horizontal elements are under consideration, this symbol will also be applicable if the element is treated as being rotated into a vertical position.

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